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Topics covered include a description of two urban runoff models, an examination of runoff quality algorithms in the SWMM, a discussion of improvements to the Extended Transport (EXTRAN) portion of the SWMM, applications of several urban drainage models in planning, analysis and design, and a comparison of the Rational Method and the SWMM. Also included are a paper on suggested methods for standardizing the process of acquiring modeling services, and a paper on urban runoff quality data collection in the Toronto, Ontario area.

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Project Officer

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Office of Environmental Processes and Effects Research
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FOREWORD

A major function of the Research and Development programs of the Environmental Protection Agency is to effectively and expeditiously transfer, to the user community, technology developed by those programs. A corollary function is to provide for the continuing exchange of information and ideas between EPA and users, and between the users themselves. The Stormwater Management Model (SWMM) users group, sponsored jointly with Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum.

This report, a compendium of papers presented at the last Users Group meeting, is published in the interest of disseminating to a wide audience the work of Group members.

Allan Hirsch
Deputy Assistant Administrator
Office of Environmental Processes
and Effects Research

ABSTRACT

This report includes fifteen papers, on topics related to the development and application of computer-based mathematical models for water quantity and quality management, presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater Management Model (SWMM) Users Group, held 10-11 January 1980 in Gainesville, Florida.

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This report has been reviewed by the Office of Environmental Processes and Effects Research, Office of Research and Development, U.S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

STORMWATER MANAGEMENT PLANNING
USING THE PENN STATE RUNOFF MODEL

by

David F. Lakatos, P.E.[1]

INTRODUCTION

Comprehensive planning for the control of stormwater runoff is a very important part of the overall development objectives for established communities, as well as those that are experiencing rapid growth at the present time. The pattern of growth and development in most areas, however, is such that many individual developments are built separately, but together they eventually form the "overall community." The stormwater management facilities for each individual development are in turn designed and constructed separately, but they must eventually function as a single hydraulic unit for the entire community. By the nature of this pattern, it can be seen that some form of "comprehensive" analysis of stormwater runoff control facilities should be performed for developments of all sizes in order to achieve optimum stormwater management on a watershed basis.

At the present time there is a significant need for more up-to-date, but readily usable, stormwater management technology for engineers who design stormwater management systems for these individual developments. These engineers are typically associated with small consulting organizations and may not have the exposure to the state-of-the-art technology that is being developed on a national level. Their applications do not generally involve a high degree of complexity, and do not involve the contract dollar amounts that are typically associated with stormwater management projects in more heavily urbanized communities. Therefore, the situation has evolved where the technological benefits of large-scale research in stormwater management are restricted to only very sophisticated applications; and the results of this research are not accessible to a group with a very definite need.

One form of technology which has evolved from years of research in the field of stormwater management is that of computer modelling. Computer simulation models are very effective tools for analyzing the effects of stormwater runoff in urban, as well as urbanizing, areas. Consequently, a large number of simulation models have been developed. Many of the existing models are very flexible in the sense that they have the ability to analyze a broad range of urban stormwater problems, such as the very complicated aspect of storm runoff quality. This degree of flexibility, while desirable in some urban applications, is often wasteful in that large amounts of computer storage space and compilation time are required for operations that may never be used. In addition to this, most efforts to modify existing models have been directed toward providing even more flexibility into the analysis routines, which typically increases overall model complexity.

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In addition to this, the transfer of state-of-the-art technology is limited for computer simulation models which can be used for "smaller-scale" stormwater management analyses (i.e., for small communities or individual developments). Many of the larger, more comprehensive models, toward which national attention as well as research and development efforts have been directed, are not practical or directly applicable for the analysis of stormwater runoff in small and mid-size communities or individual developments. The benefits of major national research efforts in stormwater management are therefore not directly accessible to a great many engineers involved with stormwater management.

This has led to the situation where stormwater management plans for small communities, and individual large developments, are prepared on a piece-meal basis. This is not a very effective technique for stormwater management in that it is not coordinated, on a municipal level, or certainly on a watershed level. The general lack of comprehensive, "watershed-level" stormwater management planning points directly to this need for a transfer of stormwater management technology on a more local level. That is, there is a need for state-of-the-art techniques and "tools" which can be used to evaluate individual stormwater management designs in a comprehensive, watershed-level framework.

Many areas are identifying a real need to evaluate storm runoff, and to provide stormwater management, on a watershed basis. This is the case in Pennsylvania where recent legislation has been passed which will require that comprehensive stormwater management plans be prepared on a watershed basis. This action points even more toward the short-term need for an analysis tool that can be used to evaluate the interactions of many small, individual storm drainage systems on a watershed basis. The Pennsyvlania Stormwater Management Act is an example of the future direction and needs of stormwater management, i.e., comprehensive and coordinated evaluations of many individual storm drainage designs in order that the overall plan is responsive to the needs of the public, as well as of the environment.

The Penn State Runoff Model was developed in response to the stormwater management needs outlined above. The objectives that were adopted for the development of the models were:

(1) To produce an urban runoff simulation model that will provide acceptable hydraulic accuracy while remaining at a level of sophistication compatible with minimum practice and data-collection time, and therefore, minimum cost.

(2) To keep the model as simple and concise as possible, and thus convenient for use for small to medium-size communities, or for individual developments.

(3) To provide a stormwater management tool for the analysis of the timing of subarea flow contributions to peak rates at various points in a watershed. This tool is known as the Peak Flow Presentation Table, and will be described in a later section of this paper.

The use of the Penn State Runoff Model is increasing for "smaller-scale" applications because of its ability to provide a cost-effective tool that can be used to evaluate state-of-the-art stormwater management for any application, regardless of its size. In addition to this, it is a model that can be cost-effectively applied on a watershed basis to "tie together" several individual separate analyses into one coordinated stormwater management plan. This capability satisfies the needs of comprehensive stormwater management planning such as is being proposed, for example, in Pennsylvania's recently-enacted Stormwater Management Act.

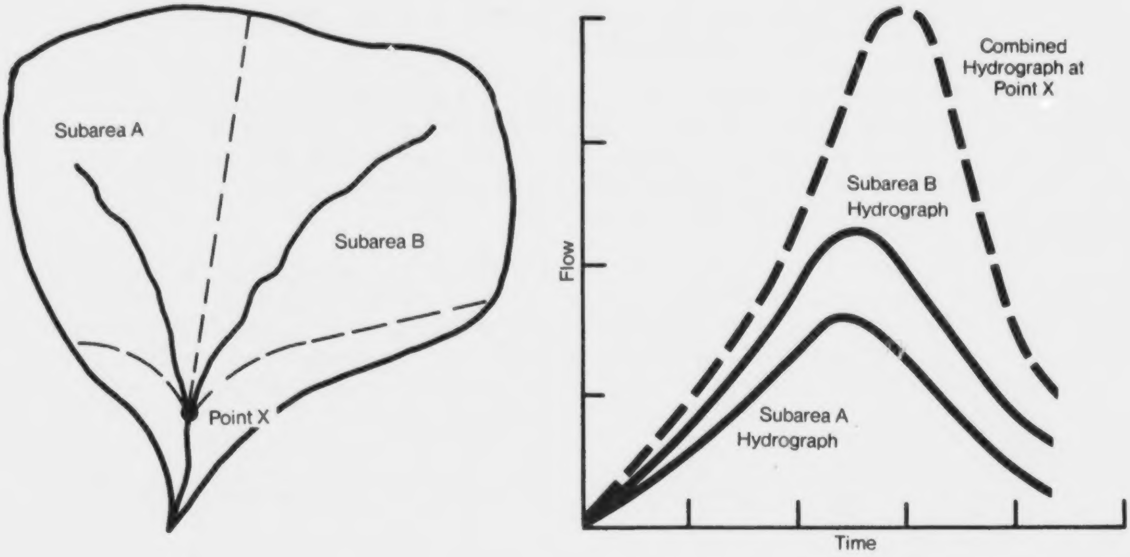
This technical paper presents a description of the Penn State Runoff Model, along with a discussion of its

- general methods of analysis
- applications, with the aid of case study examples,
- capability to serve as a tool for comprehensive watershed stormwater management planning

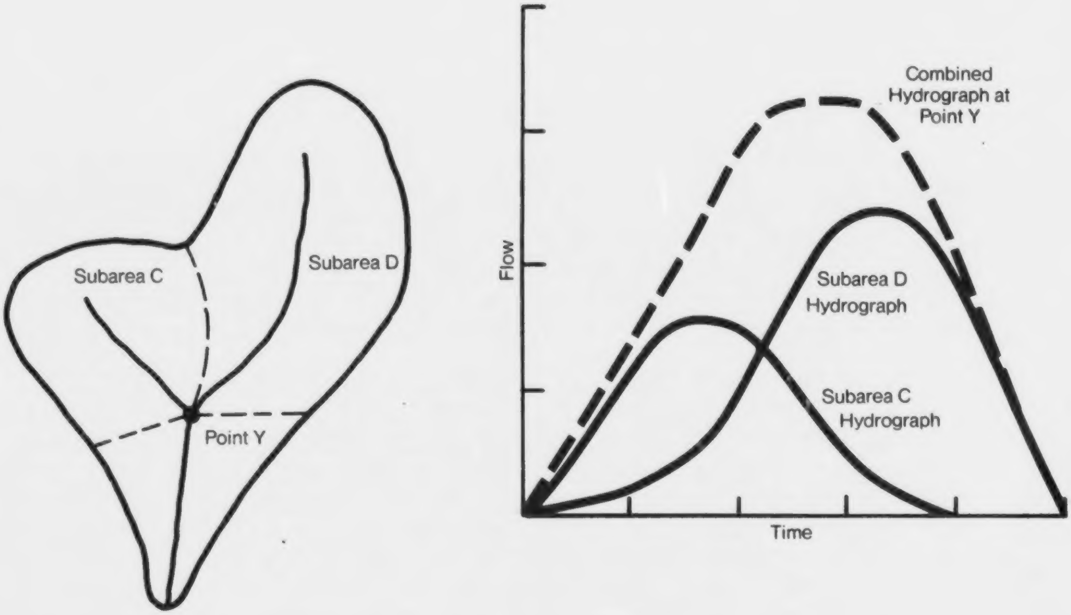
THE PENN STATE RUNOFF MODEL - A BRIEF DESCRIPTION

A visible flood flow is merely the result of the combination of smaller flows from various subareas within a watershed. Subarea flow combinations are a function of the travel times of runoff from these subareas, particularly to junction points. The relative timing of peak flows from different subareas determines the magnitude of aggregate flow downstream, which in turn is directly related to the extent of flooding that is experienced. This is illustrated conceptually in Figure 1.

It can be seen (Figure 1-A) that when the "time to peak flow" (timing) for two subareas to a junction/combination point is the same, in terms of time from the beginning of the storm event, a compounded overall peak flow occurs at the junction point (Point X). This overall peak flow is generated primarily by the simultaneous occurrence of peak flow rates from the two contributing subareas, and the magnitude of the total combined peak flow is equal to the sum of the two individual subarea peak flow rates. Conversely, if the "timing" of subarea peak flows is such that the two subarea peak flow rates do not occur at the same time (Figure 1-B), the overall peak flow rate at the junction point (Point Y) will be significantly less even though the contributing area is relatively the same.


Subarea A
Subarea B
Point X
Combined Hydrograph at Point X
Subarea B Hydrograph
Subarea A Hydrograph
Flow
Time

Subarea Peak Flows Occuring Simultaneously at Point X


Subarea C
Subarea D
Point Y
Combined Hydrograph at Point Y
Subarea D Hydrograph
Subarea C Hydrograph
Flow
Time

Subarea Peak Flows Occuring at Different Times Due to Different Lag Times

FIGURE 1
ILLUSTRATIVE EXAMPLE OF THE "TIMING" OF SUBAREA PEAK FLOWS

Development of a watershed tends to increase the stormwater runoff peak flow rate and also decreases the time to the occurrence of the peak flow rate at a given downstream point. Therefore, the development that typically takes place in a watershed can cause the peak flows for two streams, or "branches" of a watershed, to occur at the same time after the start of a storm where this was not the case under natural conditions. This situation typically causes storm runoff flooding, with its associated damages, that stormwater management seeks to alleviate. The Penn State Runoff Model was developed to provide the capability of evaluating this situation in detail, and also of analyzing and optimizing engineering alternatives for cost-effective stormwater management.

The Penn State Runoff Model is basically an urban runoff timing analysis model. A "watershed timing analysis," as it is used here, refers to a computer study of the combinations of subarea runoff flows and of their relationship to total watershed runoff for a particular storm event. This model, developed in 1976 at the Pennsylvania State University (1), was a response to the lack of an existing runoff simulation model that could be used for the analysis of the timing of subarea flow combinations. The Penn State Runoff Model can be used to analyze the effectiveness of stormwater management facilities, such as the use of stormwater detention structures, as a function of their location within a watershed drainage system.

The model was designed to be as concise as possible. Simpler methods of analyzing infiltration, of generating runoff hydrographs, and of routing flow through a drainage system were programmed into the model to reduce computer execution time and cut overall operational costs. The capability of analyzing subwatershed flow combinations through easily-interpreted illustrations was also an important objective in the development of this model.

GENERAL METHOD OF ANALYSIS

The Penn State Runoff Model simulates rainfall-runoff events on the basis of the following information:

(1) Rainfall inputs:

- rainfall hyetographs, which can vary both temporally and spatially

(2) Watershed representation:

- physical characteristics of the watershed
- conveyance system characteristics
- retention/detention basin storage characteristics

Based on this input, the model predicts the outcome of the storm in the form of runoff hydrographs, which represent:

(1) Overland flow to a drainage point

(2) Pipe flow leaving a drainage poing

(3) Surcharge flow at a drainage point

The available documentation for the model describes the calculation techniques in detail (2); the general outline of these techniques presented here provides an understanding of the basic processes being performed.

Rainfall Analysis

To allow for the spatial as well as the temporal variation of a rainfall event, the data from several recording and non-recording rain gauges can be applied to any subwatershed, and can be used to account for a system of rain gauges or for moving storm systems in a watershed. Weighting factors can be applied to rain-gauge data to provide a more accurate representation of the rainfall characteristics of particular subwatersheds. There are two techniques that are available in the model to specify, for a particular subarea, the volume and pattern of rainfall from a single design storm being used for the entire watershed. These techniques are:

(1) Thiessen-type manual weighting -- where the model user must supply a weighting variable for each recording and non-recording rain gauge being used. These variables are specified by the user for each subarea and may be determined by a Thiessen diagram or other methods. This weighting factor is then used to adjust the volume of rainfall being specified for a given time period in order that a representative value is used for each individual subarea.

(2) Pattern conserving weighting -- for this method the user must supply an x-y coordinate reading for each recording rain gauge, and also for the centroid of each subarea. The model then computes the rainfall weighting factors for each subarea as values which are inversely proportional to the square of the distances between the rain gauges and the subarea. Also, in order to avoid the situation where a distorted hyetograph is formed (excessively long duration and low intensity) by weighting several rain gauges with differing temporal rainfall distributions, the model computes the total storm depth and the temporal center of gravity of the storm for each rain gauge. To develop a design storm for each subarea, the pattern of the hyetograph from the closest rain gauge (that with the largest weighting factor) is adopted. The weighting factors are then applied to adjust the total storm depth and the storm center of gravity, thus resulting in an upward or downward scaling and a time shift of the hyetograph. This is illustrated in Figure 2.

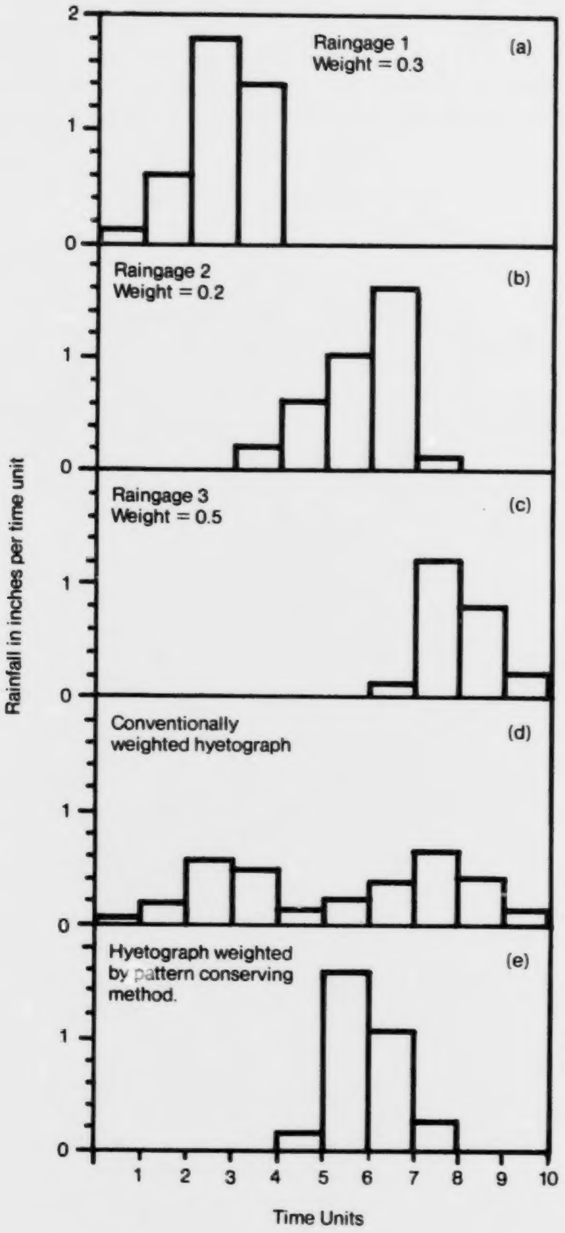
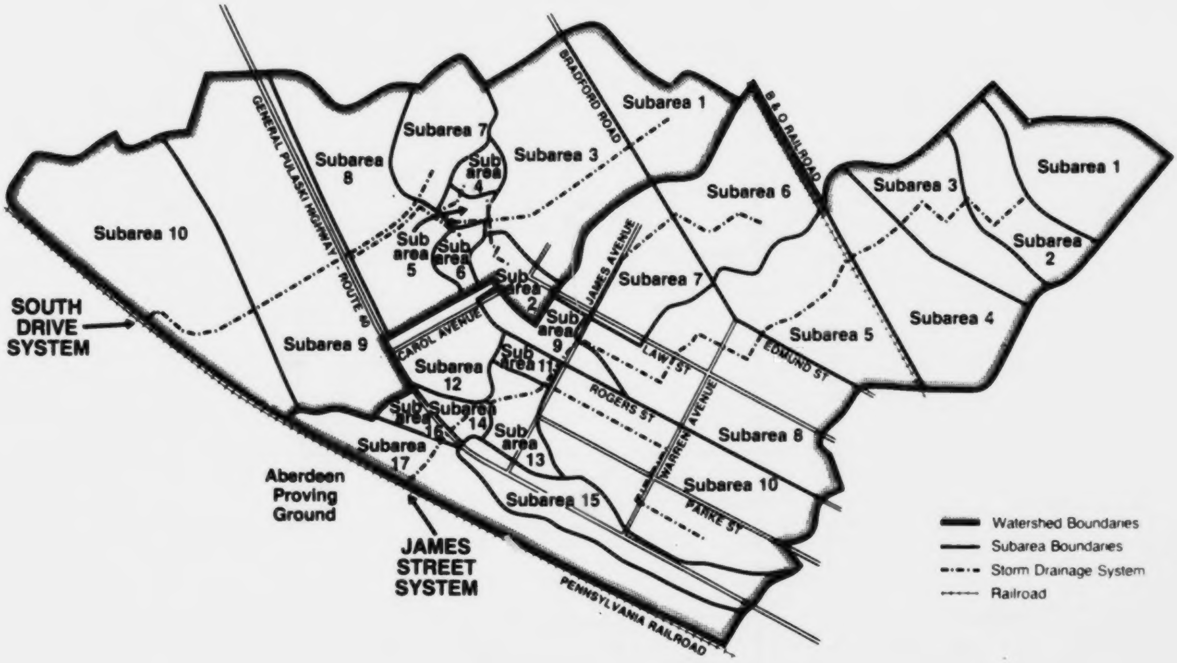


FIGURE 2
EXAMPLE OF HYETOGRAPH WEIGHTING METHODS

Watershed Characterizations

The physical system, for model purposes, consists of the water conveyance and storage systems and the physical characteristics of the watershed itself (e.g., proportions of pervious and impervious surface areas). To facilitate calculation of actual runoff, the watershed can be divided into any desired number of subwatersheds or subareas. These subareas are numbered in downstream sequence as shown in Figure 3.



GENERAL PULASKI HIGHWAY ROUTE 40
BRADFORD ROAD
B & O RAILROAD
Subarea 1
Subarea 7
Subarea 8
Subarea 3
Sub area 4
Subarea 6
Subarea 10
Sub area 5
Sub area 6
Sub area 2
JAMES AVENUE
Subarea 7
Subarea 3
Subarea 1
Subarea 2
Subarea 4
Subarea 5
SOUTH DRIVE SYSTEM
Subarea 9
CAROL AVENUE
Sub area 9
Subarea 12
Sub area 11
LAW ST
EDMUND ST
ROGERS ST
WARREN AVENUE
Sub area 16
Subarea 14
Sub area 13
Subarea 17
Subarea 8
Subarea 10
Aberdeen Proving Ground
Subarea 15
PARKE ST
JAMES STREET SYSTEM
PENNSYLVANIA RAILROAD
Watershed Boundaries
Subarea Boundaries
Storm Drainage System
Railroad

FIGURE 3
TYPICAL STORM DRAINAGE SYSTEM SCHEMATIC

Only the main sewer (or natural drainage element) is considered, and wherever a tributary joins a main drainage stem, the subarea numbering system jumps to the upper extreme of the tributary. The numbering system then proceeds in a downstream order to the next junction, where the process is repeated. Up to three incoming drainage elements are allowed to combine at any one junction, but only one outgoing element is accepted. Sewer overflow is assumed to proceed parallel with the designated drainage element to the next subarea outlet, with a travel time equal to a specified multiple of the sewer travel time.

Infiltration Calculations

Infiltration losses are estimated by a manipulation of the Soils Conversation Service (3) runoff equation into the form

$$\Delta F = \frac{S^2}{(P-IA+S)^2} \Delta P,$$

in which:

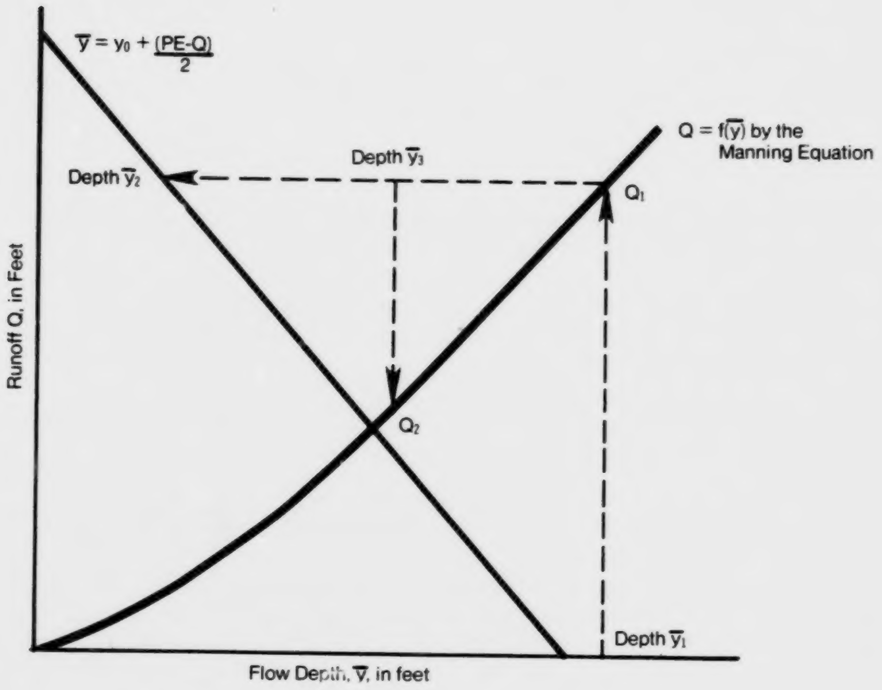
- ΔF and ΔP are infiltration and precipitation increments in inches or millimeter per unit time interval.
- S is the soil water storage capacity in the same units as F and P, as determined by the Soils Conservation Service (SCS) method.
- P is cumulative precipitation since the beginning of the storm.
- IA is the initial abstraction, assumed to be equal to 0.1S, in contrast to the SCS assumption that IA = 0.2S.

This approach for infiltration calculation was preferred over the traditionally used Horton (4) equation, principally because the Horton equation depends on infiltration or permeability parameters which cannot be quantified without specific field tests, whereas the SCS parameters are obtained from data which are mapped with some degree of consistency. Various possible alternatives of the SCS-based infiltration estimating routine are described by Aron et al (5).

Overland Flow Calculations. Overland flow is computed by the approximate kinematic wave routing method, which makes runoff a function of accumulated water depth on a subarea. The technique estimates average depth by balancing the water budget, accounting for rainfall, inflow and outflow as well as infiltration and initial losses. The continuity equation and Manning flow equation are then solved simultaneously at each calculation time step by a simple iterative method to determine depths of runoff for an area. An example of this technique is presented in Figure 4.

Drainage System Flow Calculations

The model routes the runoff hydrographs through the storm drainage system in a very simple, straightforward manner. The time that it takes for water to move from one drainage point to another through the storm drainage system is considered to be divided into a number of discrete steps. The specific number of steps for a given drainage system element is a function of the travel time in the element (e.g., a pipe or


$\overline{y} = y_0 + \frac{(PE-Q)}{2}$
$Q = f(\overline{y})$ by the Manning Equation
Depth $\overline{y}_3$
Depth $\overline{y}_2$
Q_1
Runoff Q, in Feet
Q_2
Depth $\overline{y}_1$
Flow Depth, $\overline{Y}$, in feet

RUNOFF DEPTH CALCULATION PROCEDURE:

1. Starting depth $\overline{Y}_1 = Y_0$ + Precipitation excess (PE)/2 is selected as a first approximation, from which a runoff value (Q_1) can be computed using the Manning Equation.
2. Q_1 is used in $\overline{Y} = Y_0 + (PE - Q)/2$ to compute depth $\overline{Y}_2$.
3. A third approximation $\overline{Y}_3$ is made by averaging $\overline{Y}_1$ and $\overline{Y}_2$, and the cycle is repeated by computing $Q_2 = f(\overline{Y}_3)$.
4. A final depth of runoff is converted into a runoff rate in cfs.

FIGURE 4

EXAMPLE OF ITERATIVE DEPTH-AVERAGING TECHNIQUES FOR DETERMINING RUNOFF DEPTH swale) and the time increment being used in the calculations. For each time increment, flow moves through the pipe by one step, continuing until it leaves the pipe and combines with either overland flow at a downstream subarea or pipe flow from a tributary. This process is repeated until all flow leaves the watershed.

ANALYSIS OF SUBAREA PEAK FLOW TIMING

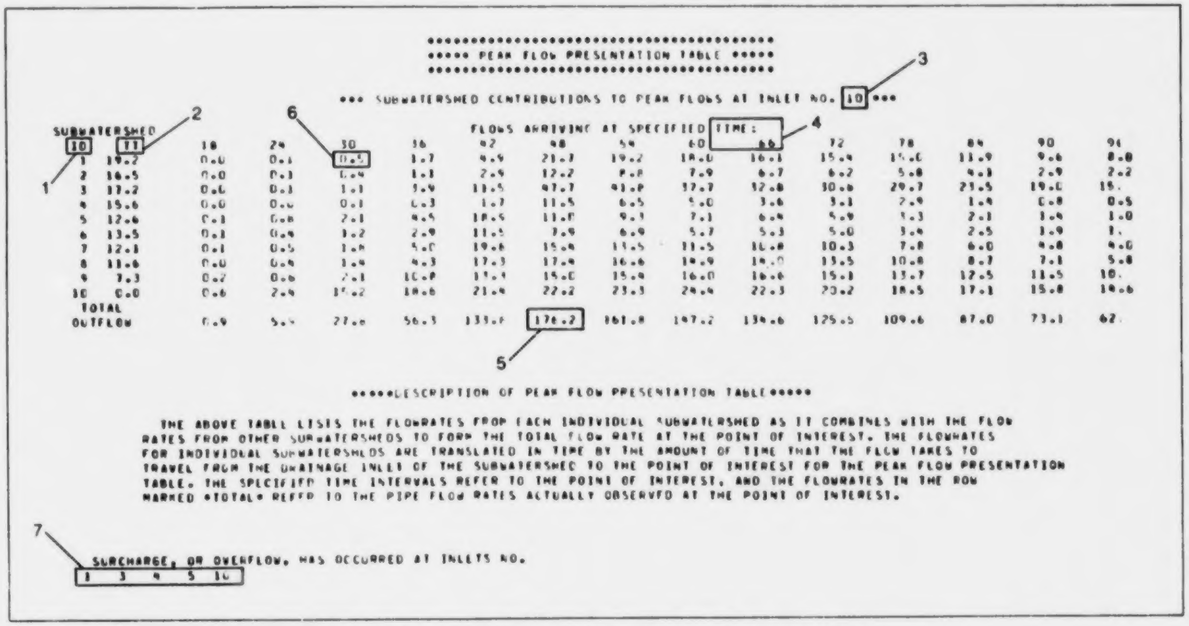
As was previously pointed out, the Penn State Runoff Model was developed to analyze and present in an easily-interpreted manner the characteristics of the timing of subarea flow contributions. That is, a total runoff volume at any particular point in the watershed drainage system, and at any particular time, is merely the sum of individual flow contributions from all the subareas draining to that point. The model was developed to have the capability of illustrating the specific characteristics of these subarea flow contributions, given that the program itself had to calculate this information as part of its normal runoff calculation routines. The particular model output which presents this information is referred to as the Peak Flow Presentation Table.

The main function of the Peak Flow Presentation Table is to display the individual runoff contributions from upstream subareas to a chosen flood-prone location, including the timing of such peak flow contributions. The flow rates presented in the Peak Flow Presentation Table reflect the travel time in the drainage system from an individual subwatershed to the particular point of interest downstream. This point of interest can be a point of observed stormwater flooding, or it can be any point in the drainage system where the analysis of the effects of stormwater runoff is desired. Presentation and review of this table enables the model user to see which subwatersheds are contributing the most critical flows to a downstream point, and to spot particularly harmful combinations of subwatershed flow rates. Thus, it points the planner to those locations chiefly responsible for the flood problem, and allows the strategic placement of stormwater management facilities.

Figure 5 contains a sample Peak Flow Presentation Table taken from model output for the analysis of a ten-subwatershed system, along with a description of the major components of the Table.

MODEL INPUT AND OUTPUT

The major operations performed by the Penn State Runoff Model are the generation of hydrographs and the routing of these hydrographs through the storm drainage system. Therefore, the general input data requirements are those which define the rainfall event, the area from which runoff will take place, and the storm drainage system that will transport the flow through the watershed. The specific input data required for the computer program includes sub-basin areas (acres), ap-



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                    ****************************************
                    ***** PEAK FLOW PRESENTATION TABLE *****
                    ****************************************

          *** SUBWATERSHED CONTRIBUTIONS TO PEAK FLOWS AT INLET NO. 10 ***

SUBWATERSHED                          FLOWS ARRIVING AT SPECIFIED TIME:
 ID    TT      18     24     30     36     42     48     54     60     66     72     78     84     90     96
  1   19.2    0.0    0.1    0.5    1.7    4.9   21.7   19.2   18.0   16.1   15.4   15.0   11.9    9.6    8.0
  2   16.5    0.0    0.1    0.4    1.1    2.9   12.2    8.8    7.9    6.7    6.2    5.8    4.1    2.9    2.2
  3   17.2    0.0    0.1    1.1    3.9   13.5   47.7   41.8   37.7   32.8   30.6   29.7   23.5   19.0   15.
  4   15.6    0.0    0.0    0.1    0.3    1.7   11.5    6.5    5.0    3.6    3.1    2.9    1.4    0.8    0.5
  5   12.6    0.1    0.8    2.1    4.5   18.5   11.8    9.1    7.1    6.4    5.9    5.3    2.1    1.4    1.0
  6   13.5    0.1    0.4    1.2    2.9   11.5    7.9    6.9    5.7    5.3    5.0    3.4    2.5    1.9    1.
  7   12.1    0.1    0.5    1.8    5.0   19.6   15.4   13.5   11.5   10.8   10.3    7.8    6.0    4.8    4.0
  8   11.6    0.0    0.4    1.4    4.3   17.3   17.4   16.6   14.9   14.0   13.5   10.8    8.7    7.1    5.8
  9    7.3    0.2    0.6    2.1   10.8 [illegible] 15.0 15.4   16.0   16.6   15.1   13.7   12.5   11.5   10.
 10    0.0    0.6    2.4   15.2   18.6   21.4   22.2   23.3   24.4   22.3   20.2   18.5   17.1   15.8   14.6
  TOTAL
 OUTFLOW          0.9    5.4   27.6   56.3  133.6  176.2  161.8  147.2  134.6  125.5  109.6   87.0   73.1   62.

                    *****DESCRIPTION OF PEAK FLOW PRESENTATION TABLE*****

     THE ABOVE TABLE LISTS THE FLOWRATES FROM EACH INDIVIDUAL SUBWATERSHED AS IT COMBINES WITH THE FLOW
   RATES FROM OTHER SUBWATERSHEDS TO FORM THE TOTAL FLOW RATE AT THE POINT OF INTEREST. THE FLOWRATES
   FOR INDIVIDUAL SUBWATERSHEDS ARE TRANSLATED IN TIME BY THE AMOUNT OF TIME THAT THE FLOW TAKES TO
   TRAVEL FROM THE DRAINAGE INLET OF THE SUBWATERSHED TO THE POINT OF INTEREST FOR THE PEAK FLOW PRESENTATION
   TABLE. THE SPECIFIED TIME INTERVALS REFER TO THE POINT OF INTEREST, AND THE FLOWRATES IN THE ROW
   MARKED *TOTAL* REFER TO THE PIPE FLOW RATES ACTUALLY OBSERVED AT THE POINT OF INTEREST.

  SURCHARGE, OR OVERFLOW, HAS OCCURRED AT INLETS NO.
  1   3   4   5  10
```

MAJOR COMPONENTS OF "PEAK FLOW PRESENTATION" TABLE:

1. Subwatershed Identification Numbers.
2. Travel Time for a Particular Subarea - amount of time that it takes for flow to travel from the particular subarea to the point of interest.
3. Point of Interest - point in the storm drainage system for which the analysis and table are being presented.
4. Time Steps in Minutes - time periods for the hydrograph at point of interest.
5. Calculated Total Flows for the Point of Interest - Includes overland flow (Total Outflow) for the subarea at the point of interest, plus upstream flow contributions.
6. Calculated flow for the particular subarea (ID), for the time when this flow reaches the point of interest.
7. Indication of those subareas where surcharging has occurred.

FIGURE 5 PEAK FLOW PRESENTATION TABLE

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proximate land slopes and overland flow widths, percentages of impervious area, roughness coefficients, and SCS curve numbers for the pervious areas, as well as pipe conveyance capacities and travel times between points in the storm drainage system. Aside from these physical data, rainfall increments per chosen time interval must be entered for at least one and at most twelve rain gauges.

The program output consists of

(1) A tabular list of all rainfall-runoff information, i.e., rainfall in inches, runoff in cubic feet per second, storage in acre-feet, for each printing time interval selected by the user.

(2) Cumulative volumes of each element of the rainfall-runoff process for the particular storm event.

(3) A listing of input data used by the program.

(4) Hydrograph plots when requested.

(5) Peak Flow Presentation Tables

PROGRAM SIZE AND REQUIREMENTS

The Penn State Runoff Model contains approximately 800 Fortran IV statements and, once compiled, requires approximately 146 K-bytes of computer memory to load and 134 K-bytes to execute. No off-line or scratch files are required. A recent execution having the following characteristics

- 67 subareas
- 365 acres (total watershed area)
- Three continuation runs of 28, 13, and 26 subareas each
- Two reservoirs

required 4.5 seconds of central processing unit time and cost approximately $6.70 at a commercial service bureau. Comparisons performed during the initial development of the model indicated that run-time costs for the Penn State Runoff Model were significantly less than those for either the HEC-1 (6) program or the EPA Stormwater Management Model (7). These comparison runs were for identical watersheds, and for output that was as similar as possible.

APPLICATIONS AND CASE STUDY USE

The Penn State Runoff Model is beginning to be applied and used by consulting engineering firms for various practical stormwater management and planning studies. Original verification of the computational routines in the model was performed using the data for the Winohocking Watershed in Philadelphia, and the Boneyard Creek Watershed in Illinois (1). A complete description of the development of the model, including its application and verification is given in a Pennsylvania State University research publication (1). A detailed description of a case study application has been prepared for several national stormwater management conferences (8, 9). These references should be consulted for specific use of the model for cost-effective stormwater management evaluations. Descriptions of input data needs, as well as the potential sources of input data, are presented in (8).

A recent application of the model for analysis of stormwater management facilities highlights is practical use for developments as well as small to medium-size communities. The application involved the analysis of an existing storm runoff detention pond for a medium-size residential development. The existing detention pool was designed using the more traditional approach of determining the pre- and post-development runoff for the 100-year design event, and then conservatively sizing the pond to be able to basically contain the difference. This resulted in a basin that had a total available volume of approximately 40 acre-feet, of which approximatey 20 acre-feet was used for a permanent wet pond.

The Penn State Runoff Model was then used to determine if there were more optimum locations for smaller detention basins which would still provide for a similar degree of stormwater management. In addition to this, the model was used to simulate the operation of the proposed basin during a design storm event. The results of this simulation illustrated the major problem associated with traditional approaches to storm runoff analysis, which do not consider either the timing of runoff flows or the detailed operation of a detention structure. That is, the simulated operation of the proposed basin illustrated that even during the 100-year design storm, only approximately 60% of the available, useful storage would be used. This is primarily because the level of detail in the analysis provided by the model allows for a "finer" evaluation of the inflow/storage utilization/outflow characteristics of the basin. This is primarily due to the relatively short calculation time steps which can be used with the modeling approach. A "hand calculation" analysis of the basin does not provide sufficient detail to optimally size the basin, and therefore a significant degree of conservatism must be used.

In addition to this evaluation of the proposed detention pond, an analysis of alternative basin locations was performed. The flexibility provided by the model allowed for the analysis of nearly a dozen alternative locations within a very short period of time. An evaluation of the results of these alternative analyses quickly provided information that was used to select the most cost-effective location for a combination of two very small basins. The total volume required was approximately seven acre-feet, and the overall reduction in total watershed peak flow was similar to that provided by the much larger basin.

This application highlights the central theme of the approach to stormwater management which utilizes the Penn State Runoff Model. This theme is basically that, by evaluating the characteristics of the timing of subarea flows using the Peak Flow Presentation Table, an optimum runoff control strategy can be adopted which is significantly more cost-effective than would be possible using more traditional calculation techniques. This theme, and the basic approach, has been tested and proven, and is primarily applicable to individual developments, a combination of developments, or a small- to mid-size community.

SUMMARY

The Penn State Runoff Model is an effective tool for analyzing cost-effective stormwater management systems for developed and developing areas. The model was developed in response to a need for such a tool for use in evaluating the state-of-the-art techniques for stormwater runoff control. A thorough verification of the program was performed as part of its development, and it has been used for several practical stormwater management applications. This paper presents an overall description of the capabilities of the Penn State Runoff Model and highlights its applicability for state-of-the-art stormwater management.

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COMPUTER MODELING FOR WATERSHED MANAGEMENT IN NORTHERN VIRGINIA

by

Hugo A. Bonuccelli[1], John P. Hartigan[2], and David J. Biggers[3]

Introduction

The basinwide impacts of urbanization are seldom adequately analyzed prior to actual development. As a result, corrective measures to address such impacts are often reparative rather than preventive in nature. The problem is even more difficult in watersheds which traverse several jurisdictions, each of which might typically administer its own land use planning and stormwater management programs with little regard for impacts which cross political boundaries. Local jurisdictions are seldom equipped with either the analytic tools or the institutional mechanisms required to address runoff-related problems which originate outside of their boundaries, even though the benefits of local stormwater management programs may be jeopardized or negated by stormwater management activities (or the lack therof) in neighboring jurisdictions.

As the regional planning agency for the Virginia portion of the Washington, D.C. metropolitan area, the Northern Virginia Planning District Commission (NVPDC) provides administrative and technical support for a number of multijurisdictional watershed management programs. These programs utilize both single-event and continuous simulation models as impact assessment tools to evaluate alternative solutions to basinwide problems.

This paper describes the development and application of the computer models used for this purpose, as well as the institutional frameworks and procedures under which ongoing impact assessment studies are performed. Accomplishments of the management programs to date are also summarized.

The Four Mile Run Watershed Management Program

Background

As shown in Figure 1, the Four Mile Run Watershed encloses portions of two counties and two cities that are located in the Virginia suburbs of Washington, D.C. As a result of intensive suburban development which occurred within the 19.5 sq mi watershed following World War II, much of the basin's natural drainage system was replaced by an elaborate storm sewer network, which was designed to transport storm flows downstream as

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FIGURE 1

GENERALIZED MAP OF FOUR MILE RUN WATERSHED

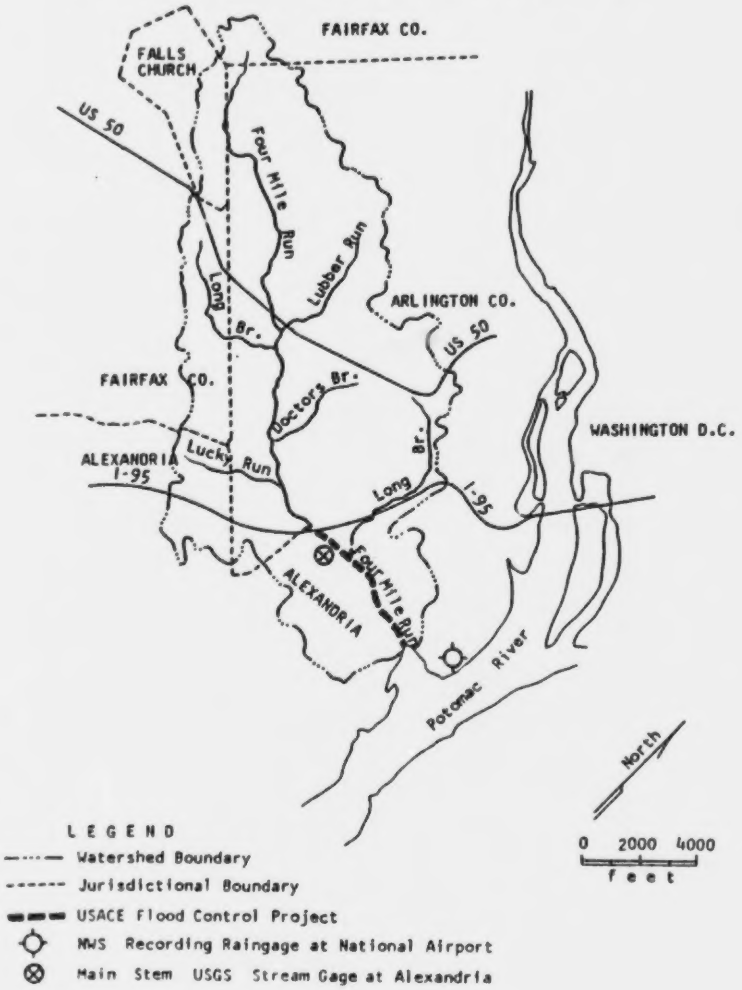
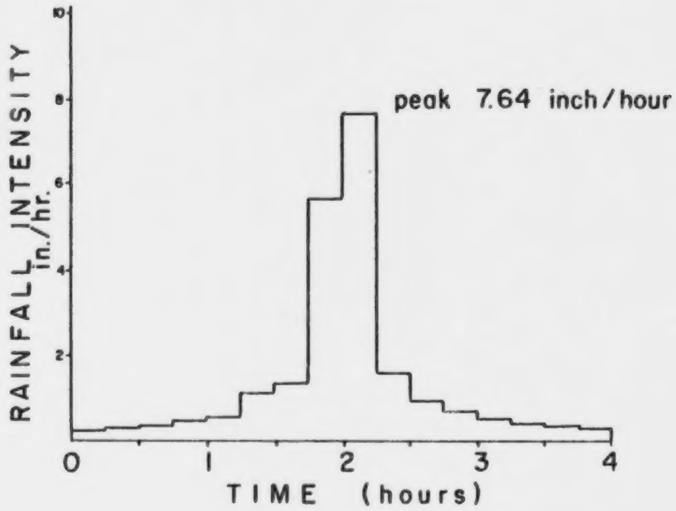

FAIRFAX CO.
FALLS CHURCH
US 50
Four Mile Run
Lubber Run
Long Br.
ARLINGTON CO.
US 50
FAIRFAX CO.
Doctors Br.
WASHINGTON D.C.
ALEXANDRIA
I-95
Lucky Run
Long Br.
I-95
ALEXANDRIA
Four Mile Run
Potomac River
North
0 2000 4000
feet
LEGEND
Watershed Boundary
Jurisdictional Boundary
USACE Flood Control Project
NWS Recording Raingage at National Airport
Main Stem USGS Stream Gage at Alexandria

FIGURE 2

WATERSHED DESIGN STORM



quickly and efficiently as possible. Approximately 37% of the land surface within the watershed is currently blanketed with impervious cover.

Residential and commercial areas located near the mouth of Four Mile Run sustained property damages totalling more than $40 million as a result of seven floods between 1963 and 1975. The periodic flash floods in this area were attributed to the cumulative impacts of sewered urban development in the basin's four jurisdictions.

In March 1974, Congress authorized a $50 million U.S. Army Corps of Engineers' (USACE) flood control project that is designed to provide protection from the 100-year streamflow event. The location of the flood control project is shown in Figure 1. In order to qualify for the USACE channelization and bridge replacement project, the Four Mile Run jurisdictions--Fairfax County, Arlington County and the cities of Alexandria and Falls Church--have been required by Congress to develop and implement a basinwide stormwater management program. This prerequisite is intended to assure that runoff from future urban development in the watershed does not produce streamflows which could impair the effectiveness of the federal flood control improvements. It is the first case in the history of USACE flood control projects in which a basinwide stormwater management stipulation has been attached to the commitment of federal funding.

In April, 1974, the Four Mile Run jurisdictions agreed to develop such a program. NVPDC was requested to provide coordination and technical support. A nine-member Technical Advisory Committee (TAC) composed of public works engineers and planners from the four jurisdictions in the watershed was formed by NVPDC to guide program development.

A two-year study was required to produce a basinwide stormwater management program that satisfied the Congressional requirement. Initial stages of the NVPDC study focused on the technical, rather than institutional requirements of a multijurisdictional stormwater management program. Following the completion of the necessary technical analyses and the development of required impact assessment tools, an institutional mechanism for implementing the stormwater management program was identified.

Development of Four Mile Run Watershed Models

After some deliberation, it was decided that the use of a computerized mathematical model of watershed hydrology would best allow the four jurisdictions to quantify and minimize flooding impacts of future development. The planning tools chosen for use in the Four Mile Run Watershed Management Program were the continuous simulation model STORM (1) and the single-event model WREM (2). Water Resources Engineers, Inc. (WRE) of Springfield, Virginia was retained by NVPDC for a two-year period to calibrate the models (3).

STORM, a relatively simple tool utilizing a modified version of the rational formula to translate hourly rainfall into runoff, was used to screen the watershed's 52-year rainfall record for critical rainstorms. Six floods that occurred during the period 1963-1972 were used for model calibration. Although STORM does not have any flood routing capabilities, simulated streamflow peaks at the main stem gaging station shown in Figure 1 were within 12% of measured values for five of the six calibration floods. The surprisingly good results of STORM applications to such a large drainage area led to the conclusion that the Four Mile Run watershed is ideally suited to STORM because its high degree of imperviousness and extensive storm sewer network have resulted in a very rapid streamflow response to rainfall and a travel time from its headwaters to the main stem streamgage of approximately one hour. Further, since STORM relied on some of the simple equations that were being applied by local public works departments for drainage system design, its successful calibration helped to alleviate anxieties about computerized hydrologic models that were shared by local public works staff.

Since the STORM analysis indicted that no historical rainfall event produced main stem flows with a recurrence interval in excess of 40 years, it was necessary to synthesize a 100-year design rainfall event for use in the Watershed Management Program. The method of Keifer and Chu (4) was used to develop the design storm shown in Figure 2. This is a very intense, 4-hour thunderstorm slightly skewed toward the receding limb, assumed to occur instantaneously over the basin. This storm is used in conjunction with model WREM to project the streamflow impacts for the Management Program.

Since the Four Mile Run versions of WREM have been described in earlier papers (5,6), only a brief summary is provided here. WREM is a second-generation version of the USEPA Stormwater Management Model. The model consists of three major programs which are executed sequentially: (a) Land Use Management (LUM) Program, which converts land use into impervious ground cover; (b) RUNOFF, which converts rainfall into surface runoff and utilizes kinematic wave routing to develop overland flow and flow in minor conduits; and (c) TRANSPORT, which routes flows through major conduits by means of a numerical methods solution to the equations of motion and continuity.

An advantage of model WREM is its sophisticated hydraulic routing capabilities, which enable it to represent open and closed conduits of varying cross-section as well as elements such as orifices and weirs. A number of runoff control measures can be simulated, including wet and dry ponds, parking lot and rooftop ponding, seepage pits, and porous pavement.

Following an extensive data collection effort, the watershed was divided into 179 subcatchments, averaging approximately 70 acres in size, drained by 97 idealized RUNOFF conduits and 101 idealized TRANSPORT channels. Three of the six floods of record were chosen for model calibration. Simulated streamflow peaks at the main stem gaging station on Four Mile Run compared well with field estimates, agreeing within 6.2% for the two most recent storms. Discrepancies noted in some of the upstream watersheds were attributed to local variations in rainfall data.

Since the physical and hydraulic parameters which describe the watershed did not require adjustment or tuning to produce satisfactory results, the analysis was changed from one of model calibration to one of model verification.

Following this exercise, model WREM was designated as the principal planning tool for the Four Mile Run Watershed Management Program. The April 30, 1975 land use pattern was chosen as the initial "baseline" for the assessment of future development impacts.

Institutional Structure of Watershed Management Program

Various alternatives were considered for the institutional framework within which the calibrated model would be utilized as a watershed management tool. After more traditional institutions such as districts and authorities were rejected by the TAC because of various technical and political shortcomings, the Four Mile Run Watershed Management Program was implemented under the "joint exercise of powers" institution (Section 15.1-21 of the *Code of Virginia*), which had not previously been utilized for water resources planning and management programs in Virginia (10). It permits two or more jurisdictions to jointly exercise any power, privilege, or authority which they are capable of exercising individually.

The Program formally began on March 31, 1977 with the signing of an interjurisdictional Memorandum of Agreement that provides for assessment of all "drainage modification projects" (i.e., any change in the watershed's land use or drainage system) with the model WREM, and requires implementation of corrective measures to offset any peak streamflow increase which might exceed the capacity of the USACE flood control works. The Memorandum also establishes a Technical Review Committee (TRC) composed of local public works officials, to oversee Program operations, and a Runoff Management Board (RMB), composed of the chief administrative officers of the four jurisdictions, to address policy matters. The TRC meets on a quarterly basis, while the RMB meets annually, or more often as required. Technical and administrative staff support for the program is provided by NVDPC. Local progress in the area of stormwater management is summarized in quarterly and annual reports which are forwarded to the USACE's Baltimore District Office for review.

Impact Assessment Procedures

Once per quarter, the local jurisdictions submit standardized forms to NVPDC summarizing all drainage modification projects (DMP's) approved during the previous quarter. Local site plan review processes have been altered to insure that appropriate data on DMP's can be generated for use in the Program. If a jurisdiction suspects that a particular project may cause negative impacts, it can request a separate assessment, funded by the developer, prior to local site plan approval.

Upon receipt of the summaries, NVPDC assigns each DMP to one of two categories for impact assessment. Projects which are less than two acres in size are assigned to a Parameter Adjustment File. These small projects are allowed to accumulate within one of the idealized model subcatchments until a sufficient number are present to warrant adjustment of the parameters describing the subcatchment in the model. Larger projects are assessed through the use of detailed representations known as SITE models, which are formulated as follows: (1) The subcatchment(s) enclosing the DMP (i.e., development project plus runoff control measure) are extracted from the watershed model. (2) A detailed SITE model of the DMP and the residual subcatchment is developed to permit an in-depth analysis of local conditions. (3) The extracted subcatchment(s) is replaced in the watershed model by hydrograph(s) generated by the detailed SITE model, and the watershed model is executed with the design storm to define projected impacts in the flood control channel. (4) If negative impacts are noted, the SITE model is used to evaluate the effectiveness of alternate control measures, and downstream impacts are checked by repeating step 3 with modified SITE model hydrographs.

Program Accomplishments

During the first 30 months of the Watershed Management Program, 223 local DMP's were reviewed. In all, 144 local DMP's have been incorporated into the watershed model. Six SITE models have been developed. The DMP's incorporated into the watershed model represent the addition of 54.7 acres of impervious cover and 37 runoff control measures providing a total of 5.6 acre-feet of detention storage.

In addition to the required quarterly review activities of the Program, a number of other impact assessment studies have been conducted with funding from private developers to determine the probable effectiveness of control measures at nine DMP's. In four cases, the downstream benefits of detention storage were documented. In four other cases, control measures were deemed unnecessary, a determination which resulted in a cost savings for the respective developers. As a result of the ninth study, two of the jurisdictions are currently designing a detention pond which will straddle a mutual boundary. A tenth impact assessment study documented the benefits of downsizing a proposed culvert, resulting in significant cost reductions for one of the participating jurisdictions.

In addition to these studies, a pair of major SITE model studies for outside agencies have been completed. The first study (7), funded by the Virginia Department of Highways and Transportation, involved the assessment of a 4.5 mile segment of Interstate Highway 66 which traverses the northern section of the watershed. The I-66 project involves the addition of approximately 100 acres of impervious surfaces and extensive stream channel improvements to the watershed. Initial watershed model assessments indicated that the original I-66 stormwater management scheme, which included one detention pond with approximately 9 acre-ft of storage capacity, focused primarily on control of runoff impacts in the vicinity of the highway, and would result in adverse downstream impacts. Following a series of watershed model evaluations of alternative runoff control levels, it was determined that a major diversion structure and two detention ponds with a total storage capacity of approximately 38 acre-ft were required to adequately address projected downstream impacts.

If the Four Mile Run Watershed Management Program had not been in existence, these supplemental controls would not have been considered.

The second study (8), funded by the U.S. Department of the Navy, involved the assessment of a major stormwater detention facility at the Henderson Hall U.S. Marine Corps station in the eastern portion of the basin. The facility, which is to be constructed with excess excavation material from redevelopment activities on the site, will not only provide approximately 17 acre-feet of detention storage, but will also result in a net savings to the Federal government of approximately $280,000, due to reduced offsite disposal costs. Model WREM assessments indicated that sizeable downstream peak streamflow reductions would be achieved by the stormwater management project. The documentation of the basinwide impacts and benefits associated with this DMP would not have been possible had the Management Program not been in operation.

Both of these studies were further noteworthy in that neither the Highway Department nor the U.S. Navy were signatories of the Memorandum of Agreement under which the Program is implemented, and thus they were not legally bound to accept the results of the model assessments.

Another special study (9) was undertaken because applications of the watershed model during the course of Management Program operations had revealed wide variations in peak flow impacts resulting from drainage modifications at various points in the basin. This watershed sensitivity study was intended to investigate such locational differences in streamflow response. A hypothetical 20-acre DMP and stormwater detention basin were simulated at twenty locations throughout the watershed and the resulting flows in the flood control channel compared.

The results of this analysis indicated that, for the Management Program design storm, the middle and upper middle portions of the watershed are most sensitive to the addition of impervious cover. The effectiveness of stormwater detention measures was likewise projected to be greatest for these locations. Additions of impervious cover to areas in the headwaters of the basin and near its mouth were projected to result in little or no adverse peak flow impacts in the flood control channel, while the provision of detention storage at these locations tended to be ineffective or counter-productive.

As a result of this analysis, the watershed was divided into stormwater management zones based on the probable effectiveness of detention storage (9). These zones are shown in Figure 3. Although larger projects will still require detailed analysis with the watershed model, these zones can be utilized by local staffs during the development plan review process to determine whether detention storage should be required at smaller sites.

The model assessments to date indicate that, as a result of the activities undertaken by the Watershed Management Program thus far, the 22,500-27,000 cfs capacity of the flood control channel is not exceeded in any idealized reach. Peak flow in the uppermost portion of the flood control channel is projected to increase by only 10 cfs in response to development since 1975, while peak flows farther downstream are projected

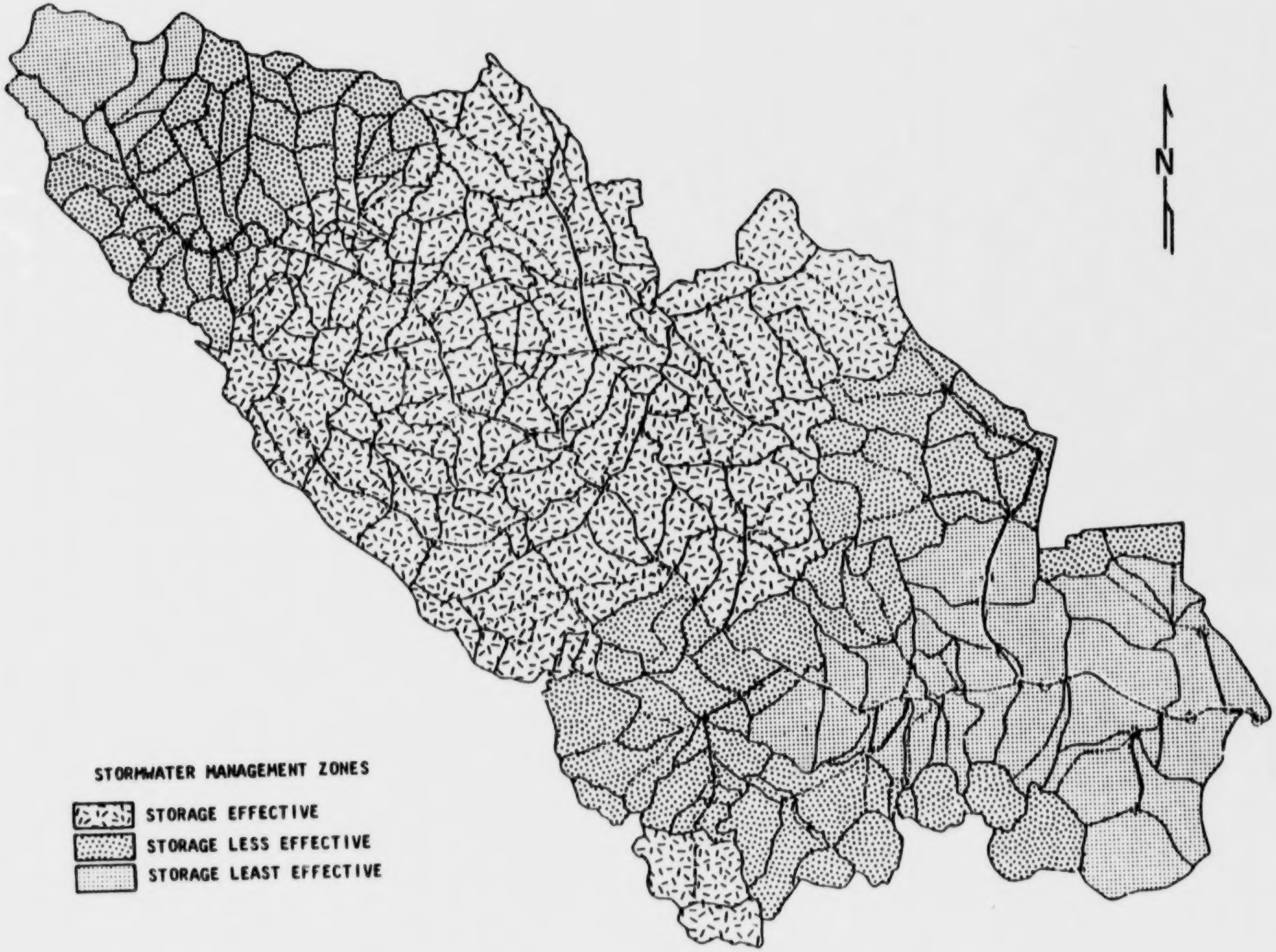

N
STORMWATER MANAGEMENT ZONES
STORAGE EFFECTIVE
STORAGE LESS EFFECTIVE
STORAGE LEAST EFFECTIVE

FIGURE 3. GENERALIZED MAP OF STORMWATER MANAGEMENT ZONES

to decrease by as much as 370 cfs. For the same development condition without stormwater runoff controls, projected increases are on the order of 400-500 cfs for every reach of the flood control channel. The recommended controls are thus projected to decrease flood control channel peak flows by 450-790 cfs (9).

Conclusion

Since more detailed discussions of both the modeling and institutional aspects of the Program have been provided elsewhere (6,10,11), the advantages and disadvantages of the adopted methodology will not be described in this paper. However, it should be noted that the incorporation of computer modeling techniques into a multijurisdictional management framework in Four Mile Run has enabled the participating jurisdictions to successfully address the basinwide flooding impacts of development in the sensitive watershed on a preventive basis, thus assuring the continued viability of the federal flood control project.

The Occoquan Basin Nonpoint Pollution Management Program

Background

The 580 sq. mi. Occoquan River Basin, which traverses four counties and two cities in the Virginia suburbs of Washington, D.C., is shown in Figure 4. The 9.8 billion gallon water supply reservoir at the mouth of the watershed was impounded in 1957 and currently serves more than 600,000 customers. It is one of the few major water supply impoundments in the Eastern U.S. that is located downstream from an urbanizing region.

In the late 1960's, classical symptoms of cultural eutrophication were observed in the reservoir. Following a one-year study (12) of water quality problems in the watershed, the Virginia State Water Control Board (SWCB) in 1971 promulgated an "Occoquan Policy" (13) for regional wastewater management which required that the jurisdictions in the basin replace eleven major secondary sewage treatment plants with a regional advanced wastewater treatment (AWT) plant situated immediately upstream from the Occoquan Reservoir. The $82 million Occoquan AWT plant began treatment operations in early summer, 1978.

The Occoquan Policy was founded on the assumption (12) that secondary wastewater treatment plants and agricultural runoff represented the major sources of plant nutrients that were degrading the quality of Occoquan Reservoir waters. Consequently, at the time of policy promulgation, it was assumed that construction of the regional AWT plant would not only eliminate wastewater sources of the contaminants but that it would also reduce nonpoint pollution loadings by accelerating the conversion of agricultural lands to suburban development.

The Occoquan Watershed Monitoring Laboratory (OWML) was created in 1972 to establish basinwide surface water quality records which could be used to gauge the effectiveness of water quality management activities. As a result, an extended record of runoff pollution loads, dry weather

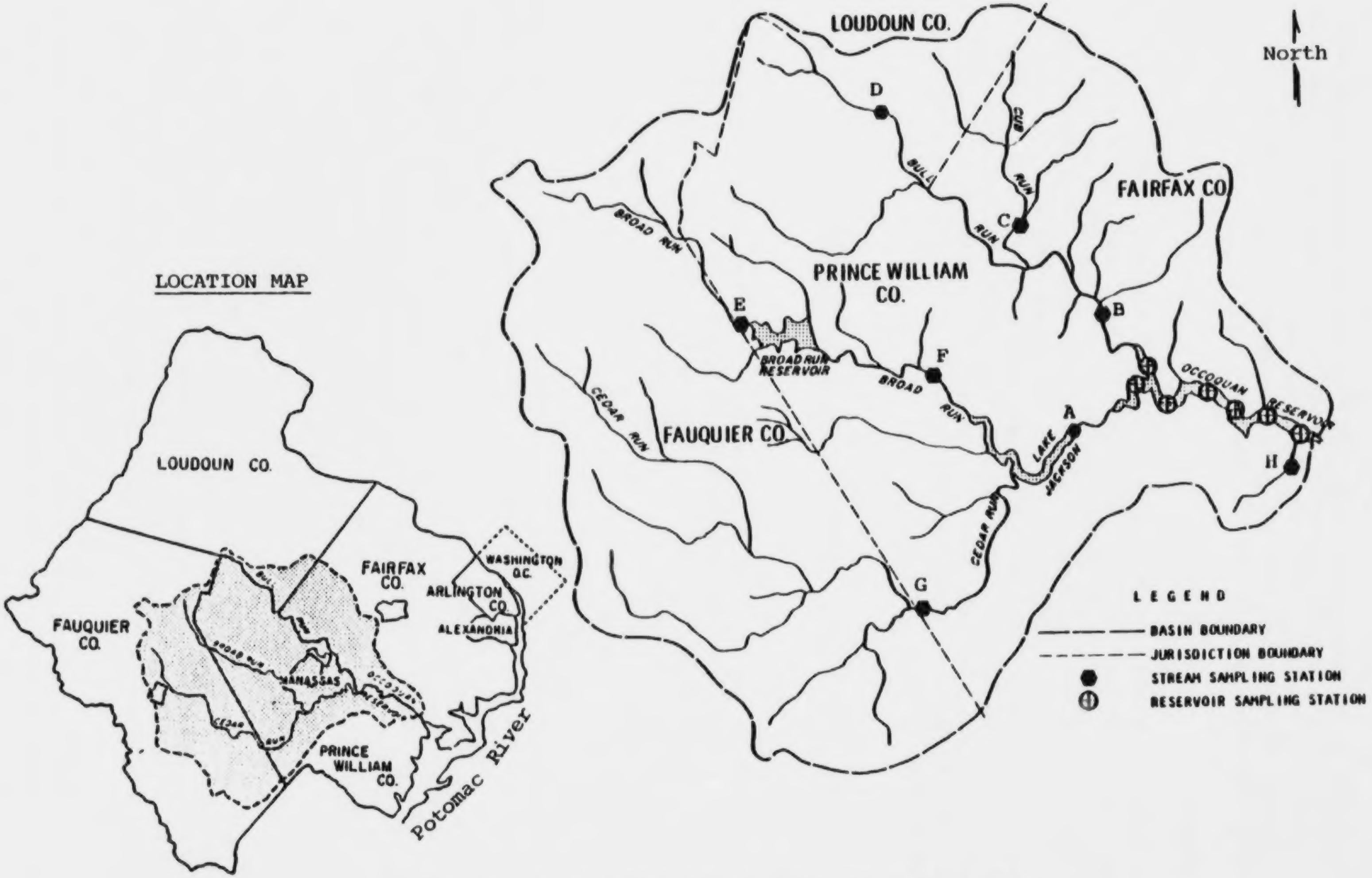

LOCATION MAP
LOUDOUN CO.
FAUQUIER CO.
FAIRFAX CO.
ARLINGTON CO.
WASHINGTON D.C.
ALEXANDRIA
MANASSAS
BULL
BROAD RUN
CEDAR RUN
OCCOQUAN RESERVOIR
PRINCE WILLIAM CO.
Potomac River
North
LOUDOUN CO.
FAIRFAX CO.
PRINCE WILLIAM CO.
FAUQUIER CO.
D
C
B
E
F
A
G
H
CUB RUN
BULL RUN
BROAD RUN
BROAD RUN RESERVOIR
CEDAR RUN
LAKE JACKSON
OCCOQUAN RESERVOIR
LEGEND
BASIN BOUNDARY
JURISDICTION BOUNDARY
STREAM SAMPLING STATION
RESERVOIR SAMPLING STATION

FIGURE 4. GENERALIZED MAP OF OCCOQUAN RIVER BASIN

flow loads, and receiving water quality has been developed for the watershed.

The development of a regional nonpoint pollution management program for the Occoquan River Basin originated with the formation of the NVDPC Occoquan Study Group in mid-1973. Composed of local elected officials and senior staff representatives of the affected jurisdictions and water management agencies, it was charged with responsibility for formulating a balanced water resources management program for the basin, which would address water resources problems that were not covered by the SWCB's 1971 Occoquan Policy. The Study Group proposed a scope of work to develop such a program which included the development of computer-based planning tools to project the water quality impacts of future land use patterns and control strategies.

In August 1975, a two-year NVPDC study based on the Study Group's recommendations was approved for federal funding under Section 208 of the 1972 Clean Water Act. In January 1978, based on the results of NVPDC's 208 Study of the Occoquan Basin and a related nonpoint pollution field study (14) conducted by NVPDC and Virginia Polytechnic Institute and State University (VPI&SU), the Occoquan Study Group formulated a basinwide nonpoint pollution management policy for inclusion in the metropolitan region's 208 plan.

The 208 Occoquan policy requires the basin's jurisdictions to develop local programs to control nonpoint pollution from future development and established a regional nonpoint pollution management program administered by a Policy Board composed of the chief administrative officers of participating jurisdictions. The program relies upon a computer-based model of the Occoquan Basin which was used to establish nonpoint pollution loading goals for the basin's jurisdictions and to monitor the multijurisdictional impacts of local land use changes and associated nonpoint pollution controls.

The NVPDC 208 study which developed the Occoquan Basin Computer Model used for impact assessments under this program relied primarily on the U.S. Environmental Protection Agency's Non-Point Source (NPS) model (15) and the Hydrocomp Simulation Programming (HSP) model (16). Hydrocomp, Inc., developer of the models, was retained by NVPDC to assist with model set-up and calibration (17).

NVPDC-VPI&SU Field Study of Nonpoint Pollution Loadings

In order to determine the nonpoint pollution loadings generated by each urban and rural-agricultural land use category, an intensive field study (14) was carried out by NVPDC and the Civil Engineering Department of VPI&SU from June 1976 through May 1977. This study was a logical extension of earlier OWML monitoring efforts (18) which had documented the significance of nonpoint pollution loadings in the basin. Runoff from twenty-one homogeneous watersheds ranging from 6 to 71 acres were monitored for the following pollutants: plant nutrients, BOD, COD, heavy metals (e.g., lead, zinc), sediment, and fecal coliforms. The study areas included four residential categories, two commercial categories, three agricultural categories, one construction site, and an undeveloped watershed which served as a control area. Thirteen of the watersheds were located in the Occoquan Basin at installations upstream from existing OWML monitoring stations. To project the impacts of dense urban development that is currently being proposed for the Occoquan Basin, eight additional monitoring stations were located in the more intensively developed Four Mile Run Watershed which is situated approximately 13 mi. east of the Occoquan Basin.

The monitoring stations relied upon automatic discrete sampling equipment that was activated by a suitable flow-measuring device at preselected flow rates. Either a natural (e.g., ephemeral stream) or artificial (e.g., storm sewer, H-flume) drainage control was used to establish stage-discharge relationships at the outlet of each watershed. Continuous recording raingages were installed in or near each study area to develop input data for computer modeling studies of each watershed and to permit periodic analyses of pollutant loadings released by rainfall.

During the 12-month field study, more than 300 runoff events were monitored and 1,300 samples were analyzed for 29 constituents. Standard statistical analyses (19) of and computer model applications (20) to the observed data produced similar relationships between land use characteristics and nonpoint pollution loading rates. The major conclusions of the NVPDC/VPI & SU field study are as follows (14):

- For plant nutrients and organics, annual unit area loading rates from urban land uses were, in general, positively related to impervious ground cover and higher than loadings from all rural-agricultural land uses, with the exception of cropland (in the case of nitrogen and phosphorus). Unoxidized nitrogen forms, which can eventually exert an oxygen demand on downstream receiving waters, represented 70%-80% of the nitrogen loadings in runoff from all land use categories.

- For heavy metals such as lead and zinc, annual unit area loading rates from urban land uses were positively related to impervious ground cover and considerably higher than loadings from all rural-agricultural land uses.

- For urban land uses, mean dissolved loadings in runoff ranged from 57%-73% of the total load for nitrogen, from 42%-55% of the total load for phosphorus, and from 6%-13% of the total load for lead. The high quantities of dissolved N and P loadings are significant from a nonpoint pollution management standpoint since this fraction is not only readily available to stimulate algal blooms in downstream receiving waters, but it is also generally not removed by stormwater detention ponds typically used to manage peak runoff.

- Air pollution is an important source of urban nonpoint pollution in the metropolitan Washington region. Atmospheric contributions of plant nutrients do not appear to be dependent on land use or distance from urban centers of poorer air quality, due to the high levels of atmospheric mixing that tend to exist during rainstorms in the region. However, since highly impervious land uses will convert larger amounts of rainfall to runoff than will land uses of lesser imperviousness, the significance of atmospheric loadings was found to vary from one urban land use to the next. Estimates of atmospheric contributions to the average annual nonpoint pollution load from urban land uses ranged from 30%-99% for nitrogen, from 20%-50% for phosphorus, and from 5%-10% for lead.

NPS Set-up and Calibration

The NPS model (15) was used to derive "land use-nonpoint pollution" relationships from the field study loading data. Nonpoint pollution loading processes simulated by the NPS model are: (a) accumulation of pollutants on the land surface and in the atmosphere during non-storm periods; (b) generation of surface sediment loads during rainfall events through raindrop impact and detachment of soil particles; and (c) overland flow transport of pollutants that have accumulated on the land surface and/or have been washed out of the atmosphere. Like most computer-based urban runoff models (1,2,16,21), the NPS model relies upon "dry weather pollutant accumulation rates" (lbs/ac/day) to represent the pollutant loading potential of urban land use classifications. Separate pollutant accumulation rates were defined for the pervious and impervious fractions of each urban land use category (20). Pollutant accumulation rates were also defined for the majority of the rural-agricultural land uses: forest land, idle land and pasture land. However, loading projections for cropland land uses, where soil loss rather than pollutant accumulation rate is the principal determinant of nonpoint pollution washoff, were based upon sediment "potency factors" (i.e., ratio of pollutant mass to sediment mass) that are multiplied by simulated sediment yield. With the long-term record of observed nonpoint pollution loadings and rainfall intensities collected during the field study, NVPDC was able to use an NPS model of each of the 21 watersheds to derive either dry weather pollutant accumulation rates or sediment potency factors for each land use category. Following calibration of hydrology parameters, each watershed model was executed with a five-minute rainfall record* for the twelve-month monitoring period and assumed values for accumulation rates and potency factors were iteratively adjusted until reasonable agreement between simulated and observed mean concentrations and loading rates was obtained.

Since pollutant contributions from urban lawn surfaces can vary from storm to storm depending upon antecedent soil moisture conditions, the pervious fraction of each watershed was extracted from the hydrology calibration data set to facilitate the characterization of pervious area loading rates. Separate models were set up for each fraction, and the pervious fraction model was executed with its five-minute rainfall record to identify those storms that produced runoff from the watershed's pervious surfaces. Calibration of the impervious fraction model involved iteratively adjusting dry weather pollutant accumulation rates until there was reasonable agreement between simulated and observed loads for storms that did not generate any runoff from the pervious fraction. Differences between observed pollutant loads and simulated impervious fraction loadings yielded calibration data for determining dry weather pollutant accumulation rates for the pervious fraction model.

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* A continuous record of five-minute rainfall volumes was used for model calibration since several of the urban watersheds were characterized by times of concentration on the order of 5-10 minutes.

Goodness-of-fit statistics for the final "land use-nonpoint pollution" relationships developed by the model calibration study are summarized elsewhere (20,22). In addition, several other technical papers (23,24,25,26,27,28,29) have been prepared by NVPDC and VPI&SU investigators to facilitate applications of the "land use-nonpoint pollution" relationships within and outside the metropolitan Washington region.

Characterizations of Best Management Practices

In order to project the water quality improvements which might result from the implementation of urban Best Management Practices (BMP's) for nonpoint pollution control, it was necessary to develop techniques to simulate the operation of such measures (30). Urban BMP's can generally be categorized as follows:

1. Source Controls: Measures that reduce the accumulation of available pollutant loads on the land surface and in the atmosphere between storm events. These include land use planning techniques, fertilizer management activities, and street/parking lot sweeping programs.

2. Discharge Controls: Measures that detain runoff for several hours and release it at a controlled rate to the drainage network.
    a. Detention Ponds: Online detention facilities that rely upon Type I sedimentation processes to remove sediment and suspended pollutants from urban runoff. Traditional flooding/erosion control designs produce "peak-shaving" facilities with relatively low detention times (e.g., 30 minutes) and considerable deviations from ideal quiescent and plug flow conditions. As a result, peak-shaving facilities achieve relatively low removal of the fine particles which are associated with the majority of the plant nutrients, BOD, and heavy metals in urban runoff. The recommended multipurpose design criteria relies upon a 24-hr detention time and quiescent conditions in a slow-release pool (50%-70% of peak-shaving storage volume) to achieve high trap efficiencies for silt particles. The use of subsurface drains to maintain a slow release pool can produce additional filtration and adsorption within the overlying soil profile. Sketches of typical outlet structures for detention basin BMP's are shown in Figures 5 and 6. To simultaneously achieve flooding/erosion control performance standards, detention basin storage capacity must be increased to approximately 1.2 times the

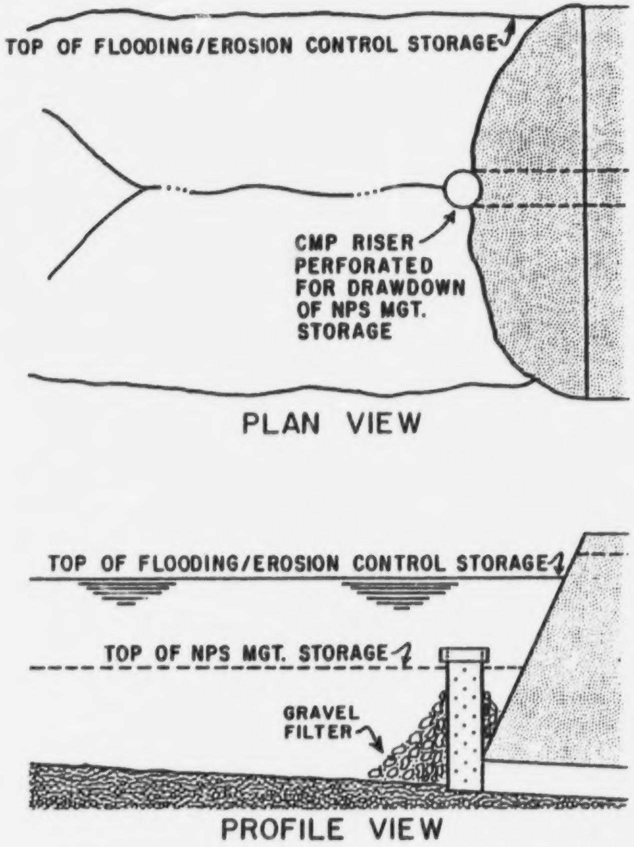
TOP OF FLOODING/EROSION CONTROL STORAGE
CMP RISER PERFORATED FOR DRAWDOWN OF NPS MGT. STORAGE
PLAN VIEW
TOP OF FLOODING/EROSION CONTROL STORAGE
TOP OF NPS MGT. STORAGE
GRAVEL FILTER
PROFILE VIEW

FIGURE 5. MULTIPURPOSE DETENTION BASIN BMP WITH PERFORATED RISER FOR NONPOINT POLLUTION MANAGEMENT

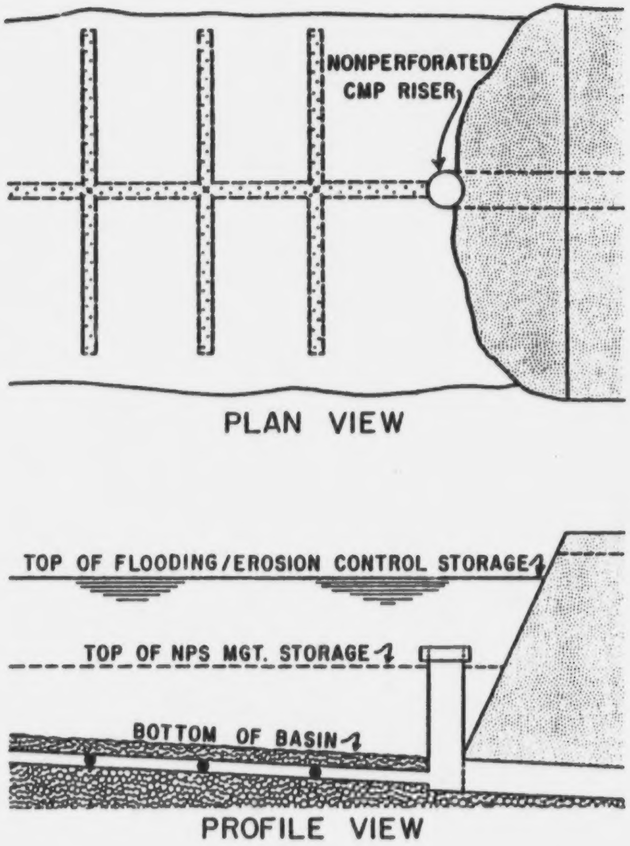
NONPERFORATED CMP RISER
PLAN VIEW
TOP OF FLOODING/EROSION CONTROL STORAGE
TOP OF NPS MGT. STORAGE
BOTTOM OF BASIN
PROFILE VIEW

FIGURE 6. MULTIPURPOSE DETENTION BASIN BMP WITH SUBSURFACE DRAINS FOR NONPOINT POLLUTION MANAGEMENT

peak-shaving level.

b. Stormwater Treatment: The addition of chemicals to stormwater detention basins can result in the removal of dissolved and colloidal pollutant loadings which would not otherwise settle out.

3. Volume Controls: Measures (e.g., Dutch drains, seepage pits, porous pavement) that reduce runoff volumes and associated loadings by diverting rainfall excess into the soil profile. Pollutant removal in the soil occurs through natural physical, chemical, and biological processes documented for "land treatment" of wastewater.

Source controls can generally be represented directly by the NPS model. For instance, street and parking lot sweeping measures can be simulated by modifying the model's dry weather pollutant decay parameters for impervious surfaces. However, in order to simulate the dynamic operation of volume and discharge controls, it was necessary to develop a STORAGE-TREATMENT submodel (17). This submodel was designed to operate in conjunction with the continuous output files generated by NPS, providing relatively simple representations of the physical, chemical, and biological unit processes that occur within stormwater management BMP's.

Since measured data documenting the pollutant removal efficiencies of typical urban BMP's is generally unavailable, average annual pollutant removal rates for BMP's serving each land use category were developed by operating the NPS and STORAGE-TREATMENT models with meteorologic data for an "average" year. The pollutant removal efficiencies derived by these BMP model applications are summarized elsewhere (30).

Occoquan Basin Computer Model

The Occoquan Basin Computer Model consists of 15 sub-basins (39 sq. mi. average) represented by the NPS model linked by 12 idealized HSP stream channels and 3 idealized HSP reservoirs. The HSP continuous simulation model was selected to serve as the receiving water submodel bacause of its ability to simulate pollutant transport and transformation processes and to project long-term impacts with the use of "pollutant concentration vs. frequency" relationships.

Model Calibration. To calibrate the NPS submodel's hydrologic parameters, simulated and measured runoff volumes and streamflow peaks at the Bull Run, Broad Run, and Cedar Run gages were compared for the period October 1970 - September 1975. Verification of the hydrology calibration results was based on comparisons of simulated and measured volumes for the period October 1967 through September 1970 at the three calibration gages and a fourth gage on Occoquan Creek. The calibration and verification results were quite good. With the exception of a single year at one of the gages, average error for the calibration period was less than 10%, with a maximum error of less than 25%. Similar results were obtained for the verification period. The discrepancies noted between simulated and recorded values approximated the expected measurement errors in rainfall and streamflow data at the respective gages.

To calibrate the HSP submodel's stream transport/transformation parameters and to verify the transferability of the "land use-nonpoint pollution" relationships produced by the NVPDC/VPI&SU field study, the NPS submodel was executed for each of the 15 sub-basins in the watershed and the resultant output time series served as input for HSP submodel executions. Simulated and measured water quality data were compared at OWML monitoring stations for the period January, 1974 - December, 1976. A pair of auxiliary computer programs were developed to aid in this calibration/verification study. The first program was used to generate line printer plots of observed and simulated "pollutant concentration vs. frequency" curves for each set of calibration parameters. As a further check, a second program which generated line printer plots and simple linear regression equations for observed and simulated nonpoint pollution loads at two free-flowing stream monitoring stations was developed. After some adjustment of instream process parameters, it was concluded that the model adequately represented land surface loadings and instream transformations of pollutants in the basin.

A detailed summary of calibration results is presented elsewhere (17,22).

Impact Assessment Studies. Long-term water quality impacts of land development patterns and BMP strategies could be projected by using the Occoquan Basin Computer Model to simulate nonpoint pollution loadings and receiving water responses produced by the 28-year (1949-1976) hourly rainfall record available at the basin's recording raingages. However, to minimize computer costs, a twelve-month period was identified which was characterized by simulated hydrologic and water quality statistics most typical of the statistics associated with the entire 28-year record (17). This "average" year (January 1976 - December 1976) was identified by operating the model for each year in the 28-year meteorologic record and comparing the annual statistics with the 28-year statistics. Projected hydrologic and water quality responses produced by operating the computer model with the average year meteorologic records are assumed to have statistics (e.g., mean pollutant concentrations, concentration vs. frequency relationships) that approximate those associated with the entire 28-year period.

Procedures for assessing the basinwide water quality impacts of various land use and water quality management decisions have been developed for use in conjunction with the Occoquan Basin Computer Model. Land use changes can be assessed by altering the NPS input files and re-executing the NPS and HSP submodels. Evaluations of structural BMP's require the development of individual land use submodels for each land use for which the BMP's are analyzed, so that the STORAGE-TREATMENT and HSP submodels can be executed with appropriate NPS washoff files. The effects of alternative wastewater discharges or streamflow diversions can be investigated by altering the appropriate point source or diversion files and re-executing the HSP portion of the model.

To derive a benchmark for assessments of urban BMP strategies, the Occoquan Basin Computer Model was used to compare the water quality impacts of existing and future land use patterns, assuming a year of average wetness and AWT discharges in accordance with the 1971 Occoquan Policy. The modeling study indicated that an uncontrolled Year 2005 land use pattern can be expected to produce a 16.1% increase in annual total phosphorus loadings delivered to the Occoquan Reservoir, a 15.0% increase in BOD loadings, and a 187.0% increase in lead loadings (31). Nonpoint pollution loadings of phosphorus were shown to be capable of producing eutrophic conditions, even in the absence of wastewater treatment plant loadings. The sizeable increase in lead loadings could produce higher accumulations of lead within Occoquan Reservoir bottom sediments which could have an adverse effect on aquatic life. In short, these modeling studies have demonstrated that high levels of wastewater treatment alone will not eliminate water quality problems in the Occoquan Basin and have confirmed the need for a balanced approach to water quality management in the watershed.

Separate model executions were carried out to characterize the benefits of each nonpoint pollution control strategy (31). Since the model runs assumed that these BMP's would only be applied to future urban development, BMP benefits are best viewed in terms of reductions in increased pollutant loadings associated with additional urbanization. The conclusions which were drawn from the BMP modeling studies are as follows (31): (a) traditional stormwater management techniques such as volume controls and detention basin controls (with modified design criteria) should be capable of maintaining Year 2005 nonpoint pollution loadings and ambient water quality at levels equivalent to or in the vicinity of existing conditions, in effect achieving nonpoint pollution management benefits that are reasonably close to those associated with land use controls which would minimize future development in the Occoquan Basin; (b) the adoption of traditional urban stormwater controls for nonpoint pollution management programs will involve only a 10%-20% increase in the total cost of a detention basin facility and no change in the total cost of a volume control facility; (c) although stormwater treatment BMP's promise to achieve the greatest reductions in nonpoint pollution loadings, the water quality benefits associated with this control measure do not appear to be great enough to offset its extremely high costs; (d) in light of (a), (b), and (c), traditional urban stormwater management BMP's appear to represent a much more cost-effective approach than urban stormwater treatment and a viable alternative to land use controls for the Occoquan Basin; and (e) in conjunction with the application of multipurpose stormwater management BMP's to future urban development, adoption of urban source control BMP's and rural-agricultural BMP's can be expected to produce nonpoint pollution loadings and receiving water quality impacts which are even lower than existing conditions.

Institutional Structure of Watershed Management Program

Based on the conclusions of NVPDC's 208 planning study, the 208 plan for the Occoquan River Basin provides for the establishment of a basinwide nonpoint pollution management program to supplement the benefits of the basin's wastewater management program. In striving to control water quality problems which are not addressed by wastewater treatment plants, the basinwide nonpoint pollution management program has as its goal: (a) the implementation of the most cost-effective nonpoint pollution mitigation techniques during the early stages of urbanization, so as to minimize the risk of irreversible water quality degradation and/or the need for costly remedial control measures at some later date; and (b) the management of nonpoint pollution loadings from agricultural lands within the basin. The management program is strictly advisory in nature, and as such, it is primarily a vehicle for fostering interjurisdictional cooperation, for providing continuing technical assistance to local staffs, and for monitoring local progress in the area of nonpoint pollution management.

The basinwide management program was established in November 1978. It is administered by a Policy board which is composed of the chief administrative officers of participating local governments and is advised by a special technical committee. Technical and administrative staff support for the management program is provided by NVPDC.

According to the provisions of the 208 plan, the Policy Board meets regularly to review local nonpoint pollution management activities, to monitor associated water quality changes with the Occoquan Basin Computer Model, to comment on the adequacy of local nonpoint pollution management efforts, to prepare quarterly reports summarizing local progress in the area of nonpoint pollution management, to review water quality data collected by monitoring agencies to determine if changes in basinwide water quality targets are warranted, and to adopt an annual operating budget for the areawide nonpoint pollution management program. The quarterly reports on local progress are forwarded to the governing boards of participating jurisdictions and agencies, the State Water Control Board, and the U.S. Environmental Protection Agency for review. The basinwide nonpoint pollution goal that has been established to gauge the progress of local management programs is minimal deterioration in surface water quality, as forecast by the Occoquan Basin Computer Model.

Program Accomplishments to Date

In the fourteen months that have passed since the basinwide nonpoint pollution management program was begun, activities have focused on the development of nonpoint pollution planning tools for local staff applications. The NVPDC staff has formulated an Urban BMP Guidebook (30) which outlines estimated BMP efficiencies and cost-effectiveness relationships for alternative design criteria. In addition, NVPDC has assisted local staff with the review of urban development proposals, the formulation of urban BMP recommendations, the evaluation of alternative local frameworks for nonpoint pollution management, and the identification of the most appropriate agricultural BMP's for the Occoquan River Basin.

Some jurisdictions have already made considerable progress in implementing nonpoint pollution controls. The Fairfax County Board of Supervisors has recently agreed to incorporate urban BMP requirements into the County Public Facilities Manual so that BMP's can be required for all new development that occurs in the Occoquan Basin. Fauquier County has relied upon its subdivision regulations to require comprehensive BMP plans for several major single family developments; in addition, the County is constructing seven major flood control/water supply impoundments, funded by the U.S. Department of Agriculture's PL-566 program, that are projected to achieve substantial nonpoint pollution management benefits as well. The City of Manassas relies upon a vacuum street-sweeping program which covers existing as well as new development. By late Winter 1979, it is anticipated that all participating jurisdictions will have selected an approach for institutionalizing urban nonpoint pollution management programs.

Conclusion

The Occoquan Basin Nonpoint Pollution Management Program allows the participating jurisdictions to determine cost-effective solutions to severe water quality problems which traverse political boundaries. The calibrated computer model is a state-of-the-art planning tool that can be used to forecast the basinwide nonpoint pollution impacts associated with various hydrologic conditions and to compare the benefits of alternative water quality management approaches. As with the Four Mile Run Watershed Management Program, the Occoquan Program enables the participating jurisdictions to successfully address the basinwide impacts of development on a preventive basis, thus fulfilling the goals of both the local jurisdictions and Section 208 of the Clean Water Act.

Other Computer Modeling Activities

Three other water quality modeling studies undertaken by NVPDC are worthy of note. All three studies rely upon the "soil texture-hydrologic model parameter" relationships, "land use-nonpoint pollution" relationships, and water quality models developed during NVPDC's Occoquan Basin study.

One of these, undertaken under a 208 planning contract with the Metropolitan Washington Council of Governments (MWCOG), utilized computer simulation techniques to rank the metropolitan region's watersheds in terms of nonpoint pollution contributions to the Potomac Estuary. NPS was used to generate "land surface washoff" (LSWO) files for various land use-soil type combinations (32). These LSWO files were then used to calculate weighted runoff loading files for each of 20 watersheds tributary to the free-flowing portion of the Potomac. HSP was then set up on the Potomac River and used to route the nonpoint pollution loads to the Potomac Estuary. By removing the washoff loads generated by each watershed one at a time, an estimate of the relative nonpoint contribution of each tributary area was obtained. Watershed contributions were ranked and the tributary areas in the top quartile were designated for intensive nonpoint pollution management investigations.

A second study, currently underway, is developing 208 Watershed Management Programs patterned after the Occoquan Basin Nonpoint Pollution Management Program for three other multijurisdictional watersheds--Goose Creek, Broad Run, and Sugarland Run. Watershed models (33), relying upon the NPS and HSP submodels, will serve as the principal planning tool for management studies of the urbanizing 490 sq mi study area. This watershed management study will be completed in July 1980.

A final modeling study (34), funded by the Virginia State Water

Control Board (SWCB), will produce nonpoint pollution loading projections for 11 watersheds (886 sq mi total) tributary to Potomac Estuary embayments. NPS and HSP submodels will be set up for each watershed. Following model calibration, the watershed models will be used by NVPDC to simulate monthly, seasonal, and annual loadings on the respective embayments, which serve as the interface between the region's free-flowing streams and the Potomac Estuary. The results of the watershed modeling study will be used by the SWCB to determine whether local nonpoint pollution management programs within the 11 watersheds might justify a relaxation of effluent standards at four advanced wastewater treatment (AWT) plants which discharge to Potomac embayments. Thus, the NVPDC study could potentially affect capital and O&M costs at AWT plants in four member jurisdictions. This watershed modeling study will be completed in December, 1980.

Summary and Conclusions

Computer modeling techniques have been successfully incorporated into a number of regional water resources management programs in Northern Virginia. The modeling results have not only been successful from a technical viewpoint, but the models themselves have been accepted as impact assessment tools in ongoing planning programs. This acceptance has enabled the participating jurisdictions and agencies to view the watersheds as integrated systems for the first time. This allows them to quantify and address basinwide impacts rather than approach water resources problems in the piecemeal, after-the-fact fashion which had previously characterized their efforts. It further helps to insure that cost-effective, balanced stormwater management programs can be implemented before the problems become economically irreversible.

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COMPARISON OF DESIGN PEAK FLOWS CALCULATED BY THE RATIONAL METHOD AND THE EPA-SWM MODEL

by

Dr. Paul Wisner*
Atef Kassem**
Philip Cheung**

> *"...we should not readily ridicule our tested knowledge and accumulated experience in face of any new illusion, but we should seek its basic concept, if there is one, and then adopt and improve it to the best for our purpose."*
>
> *Ven Te Chow (1)*

1. INTRODUCTION

The limitations of the Rational Method (RM) have been known since early in its development, and have been extensively reviewed in several recent studies (2, 3, 4). A review of Canadian practice indicated a wide discrepancy in the selection of parameters C, and the inlet time t_i used in conjunction with the Rational Method by different municipalities (2).

Despite the development and widespread application of urban hydrology models, many drainage engineers continue to use the RM. A review of storm drainage design methods, published in 1976 by the Hydraulic Research Station in Wallingford, England (4), concluded that the use of the Rational Method should continue in the U.K., at least for a limited period of time.

Several recent Canadian drainage manuals recommend the use of hydrologic models and accept with or without limitations the RM (5, 6, 7, 8).

The parallel use of the Rational Method and more sophisticated models may lead, however, to discrepancies. From the practitioner's point of view, it is of particular importance to establish whether the RM would give results which are at least consistent with a more complex model such as SWMM. A

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question which also often arises is whether the Rational Method gives results which are conservative, i.e., whether the design of storm sewers by the Rational Method has a higher safety factor. Another question is that of the maximum drainage area for which the RM can be applied. Several comparisons of the RM and hydrologic models including SWMM were based on simulations of measured flows, most of which smaller than those considered for design conditions (2, 4).

Computation of "design flows" by means of a model requires the input of "design storms" (real or hypothetical). A comparison of the RM with SWMM used in conjunction with design storms was therefore considered as being of interest and was included in a research program conducted at the University of Ottawa.

This paper presents for discussion at the SWMM Users Meeting the preliminary results.

2. PARAMETERS IN THE RATIONAL METHOD

The Rational Method for peak runoff determination is expressed by the formula: $Q = CiA$; where Q is the peak rate of runoff (cfs), C is the runoff coefficient, i is the design rainfall intensity (in/hr) for a duration which equals the time of concentration, t_c, and A is the tributary area (acres).* The time of concentration is given by the sum of travel time and "inlet time", t_i. For a given intensity-duration-frequency curve and pipe system we may assume that i is an input and t_i a parameter. This section summarizes various methods for the selection of the Rational Method parameters C and t_i. More detailed reviews of the Rational Method can be found in referances (2, 3, 4).

The runoff coefficient C lumps the effect on runoff of various factors such as imperviousness ratio, infiltration losses, depression storage, basin slope, ... etc. The simplest and most common way of selecting C is by relating it to land use characteristics of the basin. Tables which give C for various surface types and "area descriptions" can be found in the ASCE/WPCF design manual (9). More recently, attempts have been made to determine the runoff coefficient in terms of impervious-ratio which is a typical parameter in many of the models (5, 10). A simple formula of this kind is a weighed relation of the type

$$C = \frac{C_{perv} \times A_{per} + C_{imp} \times A_{imp}}{A_{per} + A_{imp}} \qquad (1)$$

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*In the metric system a conversion factor should be included. For example, for Q in m^3/s, i in mm/hour and A in ha in order to maintain the familiar C values, the RM becomes $Q = kCiA$ with $k = 0.29$.

where

C_{perv} is the C value for the pervious area,

C_{imp} is the C value for the impervious area.

Attempts have also been made to use a more realistic representation of runoff processes.

One direction (11, 12) considers the effect of rainfall intensities on the magnitude of abstractions. It assumes that for higher intensity rainfalls the effective rainfall will increase, that is the runoff coefficient will increase.

A simple relation was also developed which adjusts the runoff coefficient by multiplying it by a frequency factor "C_f". For more frequent storms (2-10 years frequency) $C_f = 1.0$, for 1/25 years storms, $C_f = 1.1$ and for 1/100 year storm, $C_f = 1.25$ (13).

Another direction is related to the fact that rainfall abstractions will decrease with time, resulting in an increase in the runoff coefficient. Graphs expressing C in terms of the time from beginning of rainfall based on the rates of infiltration can be found in the ASCE/WPCF design manual (9) and (14, 15, 16). An empirical relation, developed by Mitci and applied in Montreal, is given in Figure 1.

In most applications, a flat allowance is made for the "inlet time", t_i. In a review of U.S. and Canadian practice (4), it was found that the RM has been applied using inlet times varying between 5 minutes and 15 minutes (2, 17, 18). These values are larger than those typical for U.K. practice which range between 1 minute (19) and 4 minutes (20). Various formulae for calculating the inlet time or overland flow time by Izzard, Kirpich, Kerby, Seelye and others were compared with a relation based on the kinematic wave (3) and the results indicate significant discrepancies (Table 1).

3. DIFFICULTIES IN COMPARING THE RATIONAL METHOD AND THE SWM MODEL - PREVIOUS STUDIES

A major difficulty in comparing the performance of the RM with other models is more or less due to the subjective selection of the RM parameters. Another reason why a direct comparison is not possible is because of the different type of input.

The Rational formula does not take into account the time distribution of rainfall, and assumes a uniform rainfall.*

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*To account for variable rainfall intensities, the RM should be applied in conjunction with a time-area or isochrone procedure.

This "block rainfall" can be obtained from intensity-frequency-duration curves, which are readily available.

Models or hydrograph methods, however, require as an input a "storm profile". Storms may be of a synthetic type or real events. Application of real storms with antecedent conditions may lead to peak flow frequency relations. In the RM it is assumed that the return period of a storm is the same as that of the peak flow.

Another factor to be considered in the comparison is the routing procedure. The Rational Method substitutes routing by consideration of different rainfalls of decreasing intensities. It does not give the designer the flow history of a runoff event. It gives only peak values and not hydrographs.

In modelling procedures, the transformation of rainfall into overland runoff and channel routing is usually conducted in two distinct parts of the analysis. This separation, which represents the normal framework of analysis for a hydrologist, is not so obvious to those used with the RM in which the two phases: rainfall-runoff transformation, and routing are lumped into a single procedure.

In comparing the RM peak flows with measurements or models, some assumptions should be made to circumvent these difficulties. For example, in the comparison with measurements, the procedure indicated in Figure 2 was used in several studies (2, 21). According to this method, an average rainfall intensity used as an input to the RM is selected for a duration equal to t_c of the real storm comprising the peak intensity.

Watkins (21) used this method to calculate the peak discharges of 283 storm events observed on 12 catchments and compared them with the observed flows. The value of mean absolute error was between 10% and 20% on seven catchments and between 20% and 26% on three. For the remaining two catchments, mean errors of 47% and 100% were found.

The same procedure was used by Wisner and Clarke in a study carried out for the Canadian Urban Drainage Committee (2). The SWM model together with other models (including the Rational Formula) were tested against measurements for small watersheds. It was found out that SWMM predicted the peak flows with an error in order of ± 20%. Flows predicted by the Rational Method had a larger error (see Table 2 and Figures 3 and 4). The study concluded that the RM is not appropriate for the simulation of runoff for a real storm event. Since for some of the areas it underestimated while for others it overestimated the flows, it was considered that additional studies are required to avoid inconsistencies in design. Formula (1) was considered more adequate than runoff coefficients which do not include the imperviousness ratio as a parameter.

Comparisons with measured flows by other authors indicate different errors. Jens and McPherson (22) reported applications in Baltimore, St. Louis, Los Angeles and Oxhey which gave a mean absolute error in the prediction of peak discharge of 31.5%. Swinnerton et al (23) tested the method on 12 storms recorded on motorways and obtained an average absolute error of 83.3%. Chow and Yen (24) used the Rational Method to compute the peak discharge for four storms on the Oakdale Avenue catchment, and obtained an average error of 16%.

4. METHODOLOGY FOR THE COMPARISON UNDER DESIGN CONDITIONS

Many of the previous comparisons between flows predicted by the Rational Method, other hydrograph methods such as SWMM, and measured flows were conducted for rainfalls which have much smaller intensities than it is usually considered under design conditions. The variation of the infiltration losses, associated with these low intensity storms, may be very significant. The relatively wide scatter of the Rational Method flows for measured storms does not give the practicing drainage engineer a clue regarding the safety of a drainage system designed by means of the Rational Method.

Verification of a model under design conditions would require high intensity storms. Measurements for storms which have, for example, 5 years recurrence intervals are, however, very rare and perhaps inaccurate. Because of the good performance of a model such as SWMM for measured storms, it is however assumed that under design conditions one may also expect reasonably good predictions for the present state of the art. This does not mean a priory, that if the RM has a poorer simulation for frequent storms it may not be used with appropriate parameters and limitations for the more severe design conditions.

It was, therefore, considered that an assessment of the Rational Method as a design tool can be made by a systematic comparison with flows determined by a model such as SWMM. SWMM was selected because of its sophisticated routing routine which also has the capability of accepting hydrographs as an input. The methodology for comparison can be applied using other models and appropriate routing techniques.

Some of the Rational Method deficiencies such as lack of pipe routing are not significant for small watersheds. The analysis was therefore conducted in two steps:

1. The Rational Method parameters were first selected in order to obtain the "best fit" with SWMM for a small watershed (less than 20 acres).

2. After the selection of the Rational Method parameters for the small watershed, flow computations are extended to larger watersheds where routing effect may be more significant.

For the first step, the main parameters used in conjunction with the Rational Method are the runoff coefficient "C" and inlet time "t_i". Since the imperviousness "I" is a main parameter in SWMM, selection of a runoff coefficient C in terms of imperviousness, using formula (1) was considered appropriate. The values of C_{perv} and C_{imp} in this formula were 0.2 and 0.9, respectively.

Since the Rational Method flows are determined from intensity-frequency curves, it was also felt that for consistency of input the SWMM flows should be computed from synthetic design storms derived from these curves. The use of storm profiles of the Chicago type (5) with three levels of discretization (2, 5 and 10 minutes) are presented in Figure 5 and is discussed in the next section. The methodology described in this paper, however, can be applied for any storm profile.

5. CONSIDERATIONS REGARDING THE USE OF CHICAGO STORM PROFILES

The design storm concept has been criticized by McPherson (25) and Marsalek (26) and others for a number of valid reasons, namely:

(i) the attempt to summarize widely varying storm patterns in a single hyetograph shape;

(ii) the exclusion of antecedent conditions which may vary from storm to storm;

(iii) the return period associated with a design storm can be misleading and technically imprecise because the frequency curves which form the basis for such computations are themselves derived from different storms, in a time sequence other than the actual occurrence and often contain non-existent dummy values;

(iv) the time of concentration varies from point to point in a watershed and the use of "design conditions" obtained on the basis of a single design storm is misleading.

The alternatives to using a single design storm, on the other hand, are:

(i) to perform a long-term simulation using a calibrated computer model and recorded rainfall data;

(ii) to use a set of recorded rainstorms as being representative of a desired recurrence interval (27);

(iii) to use a series of design storms of different durations which are comparable to real storms observed from local rainfall records (27).

Arnell (28) reported the results of simulation of peak flows with various types of synthetic rainfall data and return periods and compared them with measured peak flows for a small catchment area. It was found that the theoretical Chicago distribution by Keifer and Chu (5) yielded higher flows compared to the measured ones.

Unpublished studies conducted by J.F. MacLaren Ltd. (29, 30) reported that the "Chicago" design storms of 2, 5, 10 and 25-year return period having rainfall volumes similar to those of real storms yield flow frequency curves which are comparable with the use of real storms (Figure 6 and 7).

An analysis of reports (26) and (29, 30) revealed a difference in the choice in the time step used for the discretization of the design storm. The MacLaren studies use time steps of 5 min. while the Marsalek study uses 2 min. As shown in Figure 5, the choice of time step can significantly affect the "peakiness" of a design storm. A "peakiness factor" (PF) can be defined as the ratio of the peak intensity to the average intensity of the storm.

An examination of several critical storms from the Toronto Airport records reveals that for a duration of 1-hr., the "peakiness factor" value was in the range of 2.2 and 2.8 whereas the 1-hr., 5 yr. "Chicago" yielded PF values of 5.9, 4.2 and 2.9 when discretized at 1, 5 and 10-minute time steps.

The sensitivity of design flows determined by SWMM to the "peakiness" (directly affected by discretization) of the input hyetograph, was examined in a series of systematic simulations. Tests were conducted using 1, 2 and 5-year "Chicago" storms of 1-hour duration applied to a 96-acre hypothetical catchment shown in Figure 8. The design storms were discretized to 1, 5 and 10-minute time steps. Table 3 presents the results from these simulations and shows that design peak flows modelled with SWMM are sensitive to the "peakiness factor" of the design storms.*

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*These tests were conducted by Mr. S. Gupta, graduate student at the University of Ottawa and are discussed in detail in a non-published report, "Review of Design Storms", August 1979.

Based on these results, it seems that peakiness of a design storm should be related to the "peakiness" characteristics of real storms. In general, short duration storms have a higher probability of obtaining a high peak.

Selection of a PF value for a design storm should, therefore, be related to the time of concentration (t_c) of the catchment from which the flows are being simulated. For example, for a very small catchment, with a small t_c, it may be advisable to model a short duration, high PF value design storm. These results confirm the need to "design a design storm" (27). They also show that by appropriate selection of the time step, the peakiness factor of the Chicago hypothetical storm can be modified to generate realistic flow frequency curves.

6. COMPARISON OF DESIGN PEAK FLOWS

As a first step, detailed analysis using both the Rational Method and SWMM was carried out on a small typical residential area indicated in Figure 9. The watershed which has an area of 23.3 acres (9.43 ha) is divided into 13 subareas ranging in size between 1.2 acres (0.5 ha) and 2.6 acres (1.05 ha). For such a small area, the inlet time represents a substantial portion of the time of concentration.

Comparisons between the RM flows and SWMM flows with two inlet times (5 and 10 minutes) and three levels of storm discretization (2, 5 and 10 minutes) are presented in Figure 10. For a storm with a peak intensity of 4.3 in./hr (10 mins. discretization), the RM flows are very close to SWMM flows if the inlet time is 10 minutes. On the other hand, for the higher intensity corresponding to the 2 minutes discretization, the inlet time has to be reduced to approximately 5 minutes in order to obtain RM flows in agreement with the SWM model.

These results show that for a small area the Rational Method can be calibrated to give flows which are similar to SWMM peak flows, using a unique value of C in terms of the imperviousness and an inlet time related to the peakiness factor of the storm profile or the peak storm intensity.

For the analysis of routing effects, flows were calculated by both methods (the Rational Method with 10 minutes inlet time and SWMM with 10 minutes storm profile) for two test areas: TESTVILLE A and TESTVILLE B (see Figures 11 and 12).

The drainage arrangement of Testville A is a rather theoretical layout consisting of 20 typical subwatersheds, each having an area of 20 acres; total area is 400 acres. Each subwatershed is a typical residential area with imperviousness ratio of 30% and ground slope of 2%. Testville B represents a typical new development of approximately 165 acres.

It includes a park and two school areas. The ground slope is 2%. The total development is discretized into 21 subareas with sizes ranging between 6 acres and 12 acres.

Pipe sizes were selected in order to avoid surcharges. Routing was done by means of the WRE Transport Version of SWMM. In these tests, the SWMM peak flows from the small subwatersheds are very close to the RM flows and, consequently, any difference in the results will reflect the effect of routing. Results presented in Figures 13 and 14 for TESTVILLE B indicate that as the drainage area increases, the RM will underestimate the flows. A similar trend can be observed for TESTVILLE A as shown in Figures 15 and 16.

It also seems that the configuration of the sewer system has some bearing on the results. The relatively longer and narrower system in TESTVILLE A, which means longer time of concentration, shows a larger discrepancy.

An apparent difference between the above results and the comparison with low intensity measurements (Figures 2 and 3) is that under design conditions (for a given type of storm) the RM flows are systematically biased as compared to the SWMM flows.

The comparison between the RM and SWMM flows under design conditions show that for relatively small areas (less than 100 acres) the differences are small and may not be economically significant. This confirms that the RM, if properly applied, is an adequate tool for determination of peak design flows.

It may, however, be desirable to apply the RM in such a way that flows are in better agreement with models such as SWMM. Several possibilities will be briefly discussed:

1. an increase in runoff coefficient "C": In fact, some drainage manuals have recommended for residential areas C values which are slightly higher than those computed by formula (1). This would lead to overestimation of flows for the upper small areas. However, the effect on the selection of pipe sizes may not be significant.

2. runoff coefficient "C" variable with the time of concentration: It is possible to consider an increase of the C value in terms of the time of concentration. The trend shown in Figure 17 would support this approach. Results based on Figure 1 developed by Mitci (15) are shown in Figures 18 and 19 for TESTVILLE A and TESTVILLE B, respectively. For TESTVILLE B the RM flows computed by Mitci relation are smaller than those computed with a constant C. This can be explained by the low time of concentration for

TESTVILLE B. On the other hand, for TESTVILLE A, the RM flows computed using Mitci relation are closer to SWMM flows, but still remain systematically smaller. With more systematic testing, a new relation could be developed.

7. CONCLUSIONS

Previous comparisons against measurements show not only errors but also inconsistencies in predicting the peak flows by the RM. Most measurements correspond to rainfalls with low intensities, where antecedent conditions, time-variation of infiltration and other losses may be very significant.

Through systematic comparisons between the RM and SWMM under design conditions described in this study, it was found that the Rational Method may give consistent results:

a) For a given design storm and for small areas (less than 20 acres), the RM parameters can be selected in order to obtain flows which are in close agreement with those determined by SWMM. The runoff coefficient C can be expressed in terms of imperviousness by a simple relation (1). The "equivalent inlet time" varies with the peakiness of the design storm.

b) For larger watersheds, where requirements for routing can be significant, flows determined by the RM show a systematic bias when compared to SWMM flows. For the examined configurations and storm profiles, the RM flows were consistently smaller with the difference increasing with the area of the watershed or the time of concentration.

c) The difference between the flows determined by the RM and SWMM varies with the shape of the watershed and the configuration of pipe system.

d) For watersheds of 100 acres and less, a slight increase in C value would bring the RM flows very close to SWMM flows for the downstream part. The consequent overestimation of flows for the upper small area may not be economically significant.

e) Refinement for larger areas would consist in varying C in terms of the time of concentration. Application of relations developed by Mitci (15) resulted in better agreement of the RM and SWMM flows. However, the RM flows were still smaller than the SWMM flows.

f) The selection of design storm profile is important not only for studies with SWMM but also for applications of the RM (selection of an appropriate "inlet time").

These results seem to confirm that with careful selection of parameters, the RM is a good tool for design purposes of conventional storm sewer systems. The limiting size of the watershed can be increased if a variable runoff coefficient is adopted (e.g. C increasing with the time of concentration).

Advantages in using SWMM for design of storm sewers result mainly if surcharge and storage are analyzed. An example of a design problem which cannot be studied by the RM is the dual storage system, which is based on the principles of dual drainage (pipe and street flows), runoff control, and inlet control. Such an application would require sizing of storage elements (e.g., underground storage and surface storage), consideration of restricted outlets, backwater effects and overflows. The economic advantages and design aspects of this solution have been presented in another report (31).

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TABLE I

COMPUTATION OF TIME OF CONCENTRATION (MINUTES) BY SIX DIFFERENT METHODS (Reference 3)

ASSUMING:	PAVED	GRASSED-PERVIOUS
MANNINGS N	0.013	0.350
IZZARDS CR	0.013	0.060

L= 100. FT.

	PAVED							GRASSED-PERVIOUS						
I= 0.50 IN/HR	C= 0.82	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????	C= 0.40	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????
S=0.01		21.3	2.8	7.4	1.3	8.2	5.0		155.5	12.8	82.1	1.3	11.7	12.6
S=0.02		16.9	2.3	6.0	1.0	6.5	4.0		123.4	10.9	66.7	1.0	9.3	10.0
S=0.03		14.8	2.1	5.3	0.9	5.7	3.5		107.8	9.9	59.0	0.9	8.1	8.7
S=0.04		13.4	2.0	4.9	0.8	5.2	3.2		98.0	9.3	54.2	0.8	7.4	7.9
S=0.05		12.5	1.9	4.6	0.7	4.8	2.9		91.0	8.8	50.7	0.7	6.9	7.4
S=0.06		11.7	1.8	4.3	0.7	4.5	2.8		85.6	8.4	48.0	0.7	6.5	6.9
S=0.07		11.1	1.7	4.1	0.7	4.3	2.6		81.3	8.1	45.8	0.7	6.1	6.6
I= 1.00 IN/HR	C= 0.86	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????	C= 0.38	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????
S=0.01		13.3	2.8	4.7	1.3	8.0	4.3		101.9	12.8	55.9	1.3	12.0	12.9
S=0.02		10.6	2.3	3.9	1.0	6.3	3.4		80.9	10.9	45.4	1.0	9.5	10.3
S=0.03		9.3	2.1	3.4	0.9	5.5	3.0		70.7	9.9	40.2	0.9	8.3	9.0
S=0.04		8.4	2.0	3.1	0.8	5.0	2.7		64.2	9.3	36.8	0.8	7.6	8.2
S=0.05		7.8	1.9	2.9	0.7	4.7	2.5		59.6	8.8	34.5	0.7	7.0	7.6
S=0.06		7.3	1.8	2.8	0.7	4.4	2.4		56.1	8.4	32.6	0.7	6.6	7.1
S=0.07		7.0	1.7	2.6	0.7	4.2	2.3		53.3	8.1	31.2	0.7	6.3	6.8
I= 1.50 IN/HR	C= 0.87	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????	C= 0.35	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????
S=0.01		10.4	2.8	3.7	1.3	7.9	4.1		82.6	12.8	46.0	1.3	12.5	13.5
S=0.02		8.2	2.3	3.0	1.0	6.3	3.3		65.6	10.9	37.4	1.0	10.0	10.7
S=0.03		7.2	2.1	2.7	0.9	5.5	2.9		57.3	9.9	33.1	0.9	8.7	9.4
S=0.04		6.5	2.0	2.4	0.8	5.0	2.6		52.1	9.3	30.4	0.8	7.9	8.5
S=0.05		6.1	1.9	2.3	0.7	4.7	2.4		48.3	8.8	28.4	0.7	7.3	7.9
S=0.06		5.7	1.8	2.2	0.7	4.4	2.3		45.5	8.4	26.9	0.7	6.9	7.4
S=0.07		5.4	1.7	2.1	0.7	4.2	2.2		43.2	8.1	25.7	0.7	6.6	7.1
I= 2.00 IN/HR	C= 0.88	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????	C= 0.33	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????
S=0.01		8.7	2.8	3.1	1.3	7.9	4.0		71.3	12.8	40.1	1.3	12.9	13.8
S=0.02		6.9	2.3	2.5	1.0	6.3	3.1		56.6	10.9	32.6	1.0	10.2	11.0
S=0.03		6.0	2.1	2.2	0.9	5.5	2.7		49.5	9.9	28.8	0.9	9.0	9.6
S=0.04		5.5	2.0	2.0	0.8	5.0	2.5		45.0	9.3	26.5	0.8	8.1	8.7
S=0.05		5.1	1.9	1.9	0.7	4.6	2.3		41.7	8.8	24.7	0.7	7.6	8.1
S=0.06		4.8	1.8	1.8	0.7	4.4	2.2		39.3	8.4	23.4	0.7	7.1	7.6
S=0.07		4.5	1.7	1.7	0.7	4.1	2.1		37.3	8.1	22.4	0.7	6.8	7.2
I= 2.50 IN/HR	C= 0.89	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????	C= 0.32	IZZARD	KERBY	KINEMATIC	KIRPICH	SEELYE	????
S=0.01		7.6	2.8	2.7	1.3	7.9	3.8		63.1	12.8	35.7	1.3	13.1	14.0
S=0.02		6.0	2.3	2.2	1.0	6.2	3.0		50.1	10.9	29.0	1.0	10.4	11.1
S=0.03		5.3	2.1	1.9	0.9	5.5	2.6		43.8	9.9	25.7	0.9	9.1	9.7
S=0.04		4.8	2.0	1.8	0.8	5.0	2.4		39.8	9.3	23.6	0.8	8.3	8.8
S=0.05		4.5	1.9	1.7	0.7	4.6	2.2		36.9	8.8	22.0	0.7	7.7	8.2
S=0.06		4.2	1.8	1.6	0.7	4.3	2.1		34.7	8.4	20.9	0.7	7.2	7.7
S=0.07		4.0	1.7	1.5	0.7	4.1	2.0		33.0	8.1	19.9	0.7	6.9	7.3

Table 2

Performance of the RM and SWM Model as Assessed by the Canadian Review of Urban Drainage Practices (2)

Catchment	Area (acres)	% Imp	No. of Storms	Rational Method			SWMM		
				λ	σ	ε	λ	σ	ε
Oakdale Avenue (Chicago)	12.95	45.8	14-17	1.4	0.58	46	1.08	0.20	19
Northwood (Baltimore)	47.4	68	14	0.97	0.32	21	1.18	0.51	33
Gray Haven (Baltimore)	23.3	52	10-14	0.91	0.22	20	1.18	0.22	19
Calvin Park Kingston, Ontario	89.4	27	10-13	1.63	0.28	63	1.20	0.22	28

* These results were rejected because of inaccurate date
λ Signifies the ratio computed/measured peak discharge
σ Signifies the standard deviation of the individual values about
ε Signifies the mean absolute percentage error

TABLE 3 - SENSITIVITY OF SWMM (RUNOFF) TO PEAKINESS OF A CHICAGO STORM

RETURN PERIOD (YRS)	TIME STEP OF DISCRETIZATION (MINUTES)	PEAK FLOW (IN./HR.) AREA (ACRES) 12	64	96
	1	1.36	1.09	0.94
1	5	1.01	0.92	0.83
	10	0.79	0.75	0.72
	1	1.57	1.28	1.12
2	5	1.19	1.09	0.98
	10	0.93	0.88	0.85
	1	2.01	1.69	1.49
5	5	1.53	1.44	1.32
	10	1.17	1.13	1.11

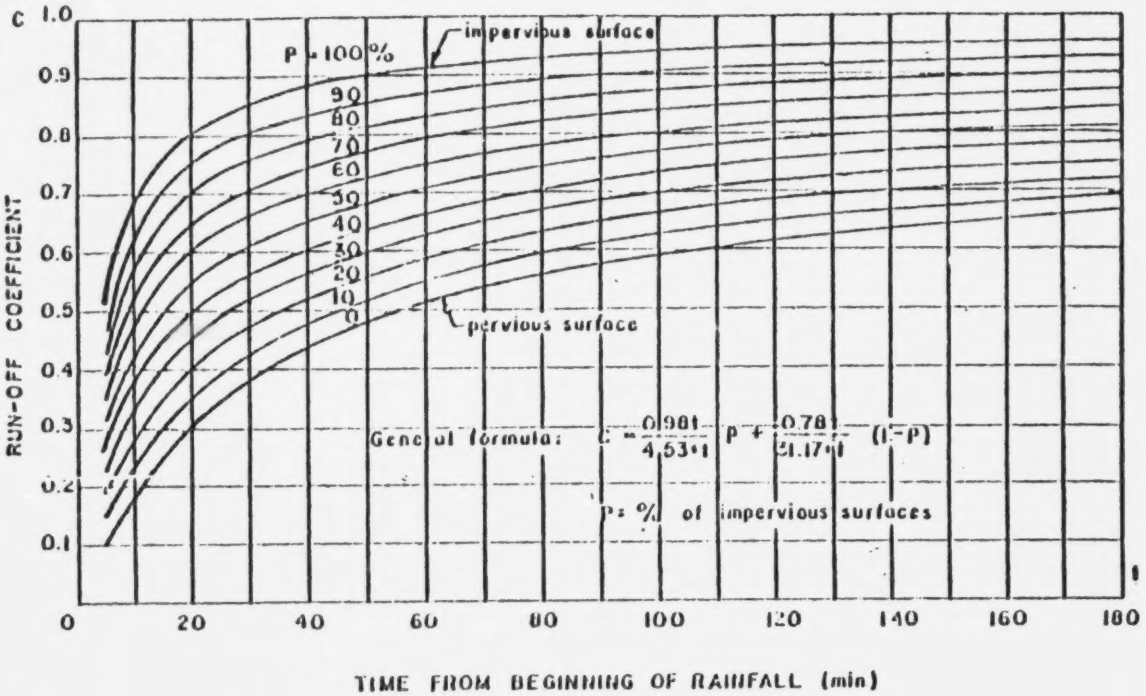
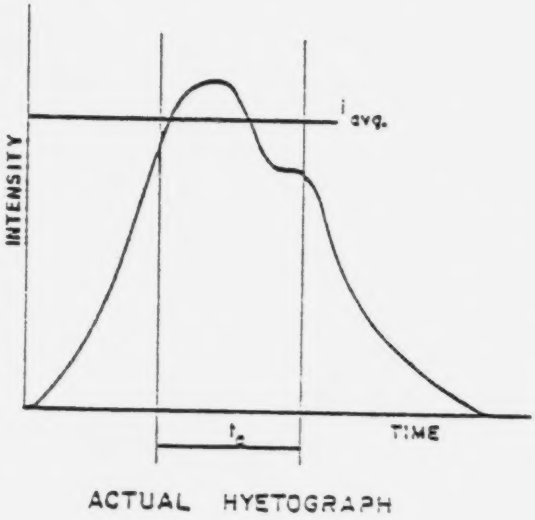


Figure 1: C as a Function of Time (from Ref. 15).



1. t_c DETERMINED AT OUTLET
2. MAX. AVG. INTENSITY FOUND FOR t_c FROM RECORDED STORM HYETOGRAPH.
3. C, A FROM PHYSICAL DATA
4. Q MAX. = C i A

Figure 2: Estimation of Peak Rainfall Intensity from Recorded Rainfall for RM Peak Flow Computation. (Ref.2)

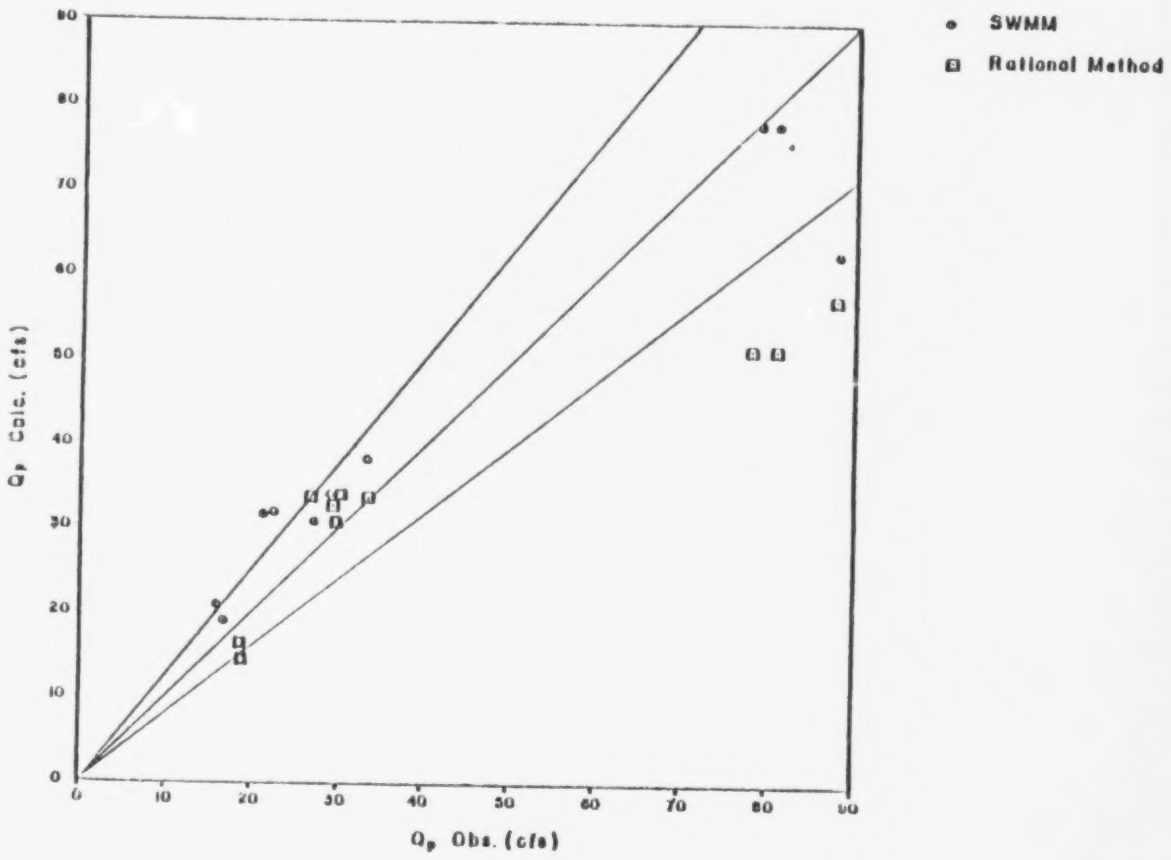


Figure 3 Comparison Of Measured And Calculated Peak Flows For The Gray Haven District In Baltimore (Data From Reference [2]).

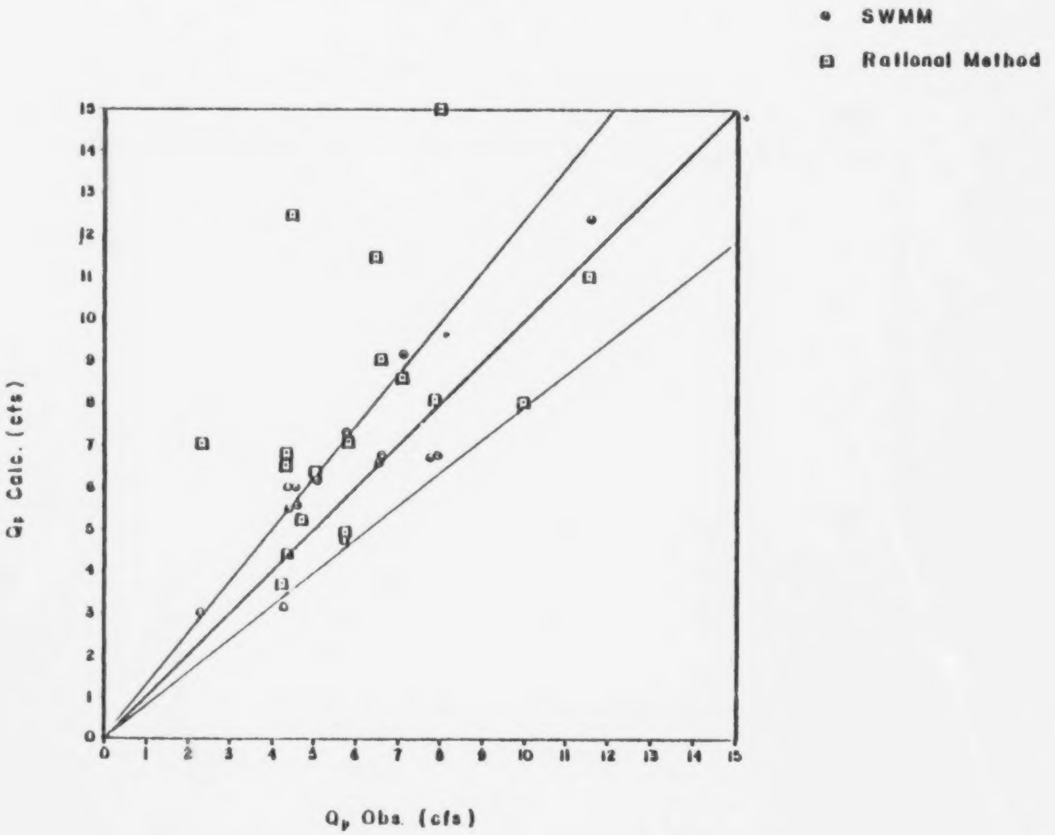


Figure 4 Comparison Of Measured And Calculated Peak Flows For The Oakdale District In Chicago (Data From Reference [2]).

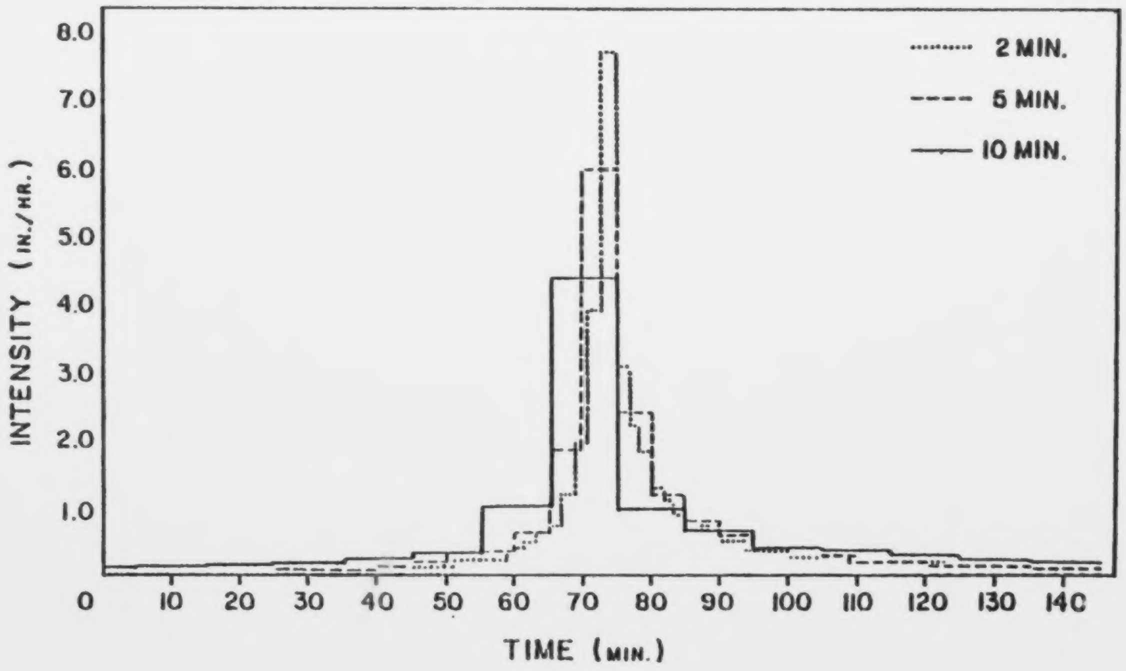
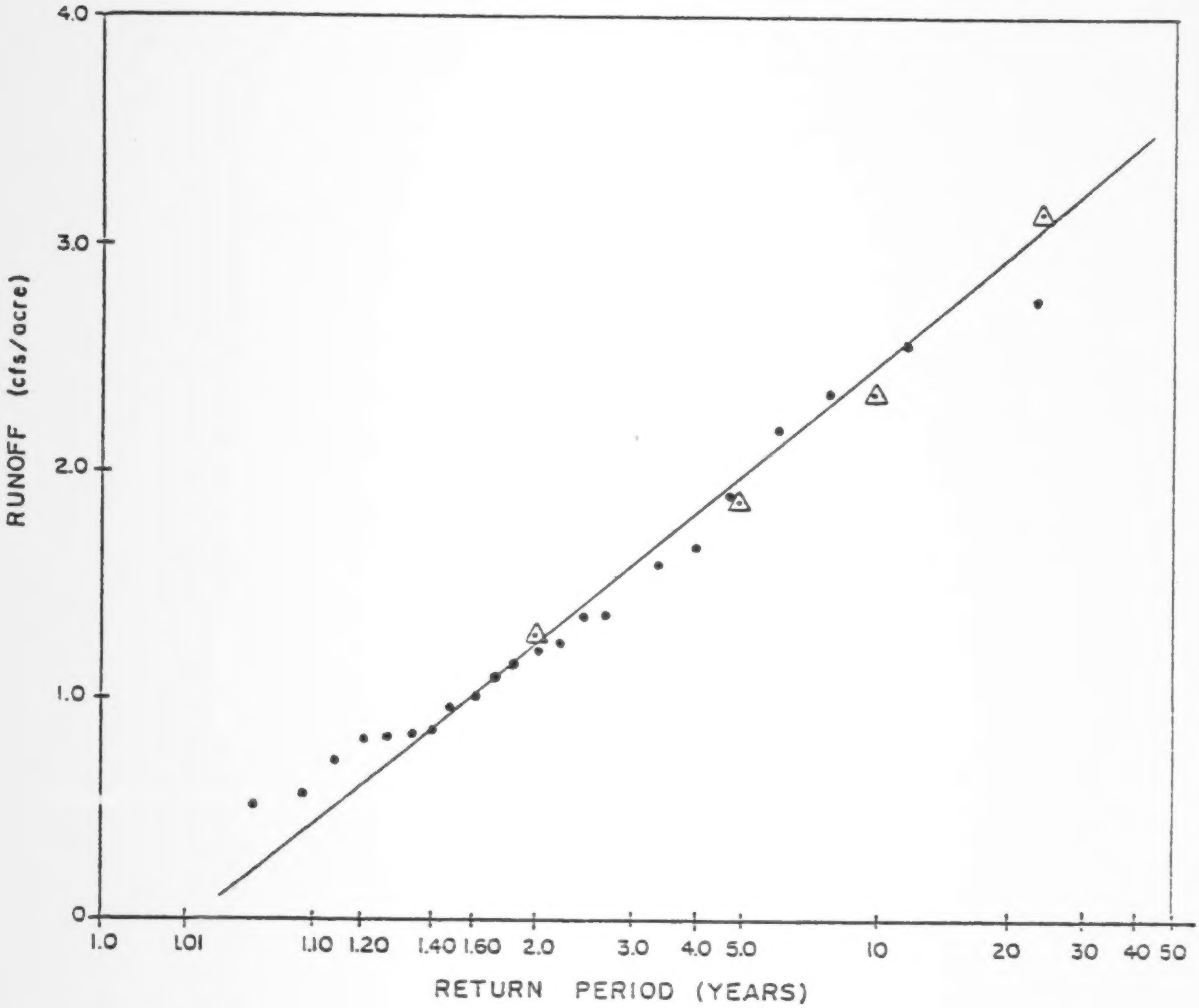


Figure 5: 5 Years Intensity-Duration-Frequency Curve and the Corresponding Chicago Design Storm (10 mins., 5 mins., and 2 mins. discretization).



LEGEND

- • REAL STORM
- △ DESIGN STORM

1 cfs/acre = 0.070 m^3/s hectare

FREQUENCY OF EXCEEDENCE
VS
RUNOFF

Tc= 15 min.

FIGURE 6 (Reference 29)

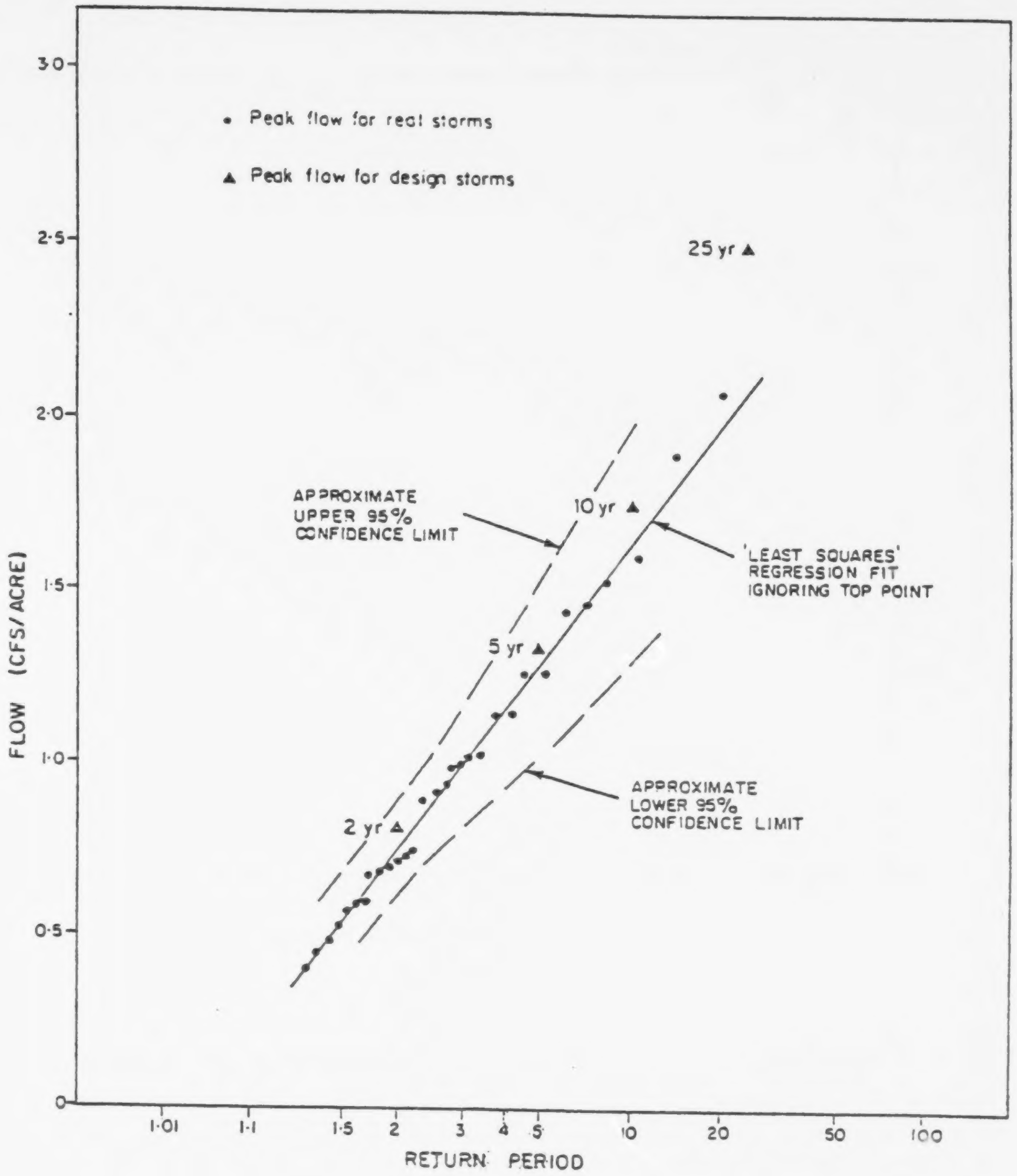


FIGURE 7 (Reference 30)

FLOW FREQUENCY ANALYSIS FOR REAL STORMS, LOCATION 6801
(14·2 ACRES)

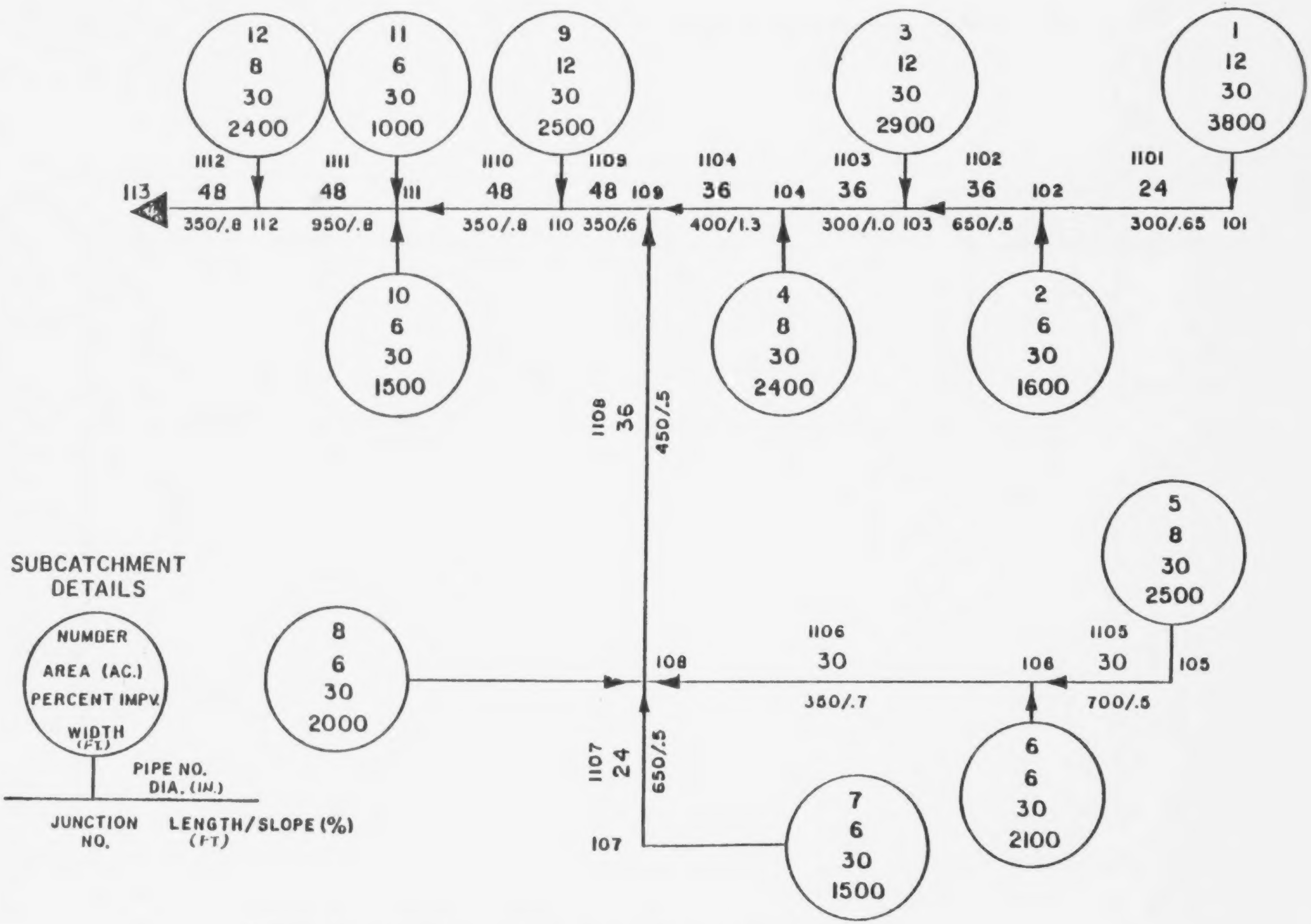

1
12
30
3800
2
6
30
1600
3
12
30
2900
4
8
30
2400
5
8
30
2500
6
6
30
2100
7
6
30
1500
8
6
30
2000
9
12
30
2500
10
6
30
1500
11
6
30
1000
12
8
30
2400
1101
24
300/.65
101
1102
36
650/.5
102
1103
36
300/1.0
103
1104
36
400/1.3
104
1105
30
700/.5
105
1106
30
350/.7
106
1107
24
650/.5
107
1108
36
450/.5
108
1109
48
350/.6
109
1110
48
350/.8
110
1111
48
950/.8
111
1112
48
350/.8
112
113
SUBCATCHMENT DETAILS
NUMBER
AREA (AC.)
PERCENT IMPV.
WIDTH (FT.)
PIPE NO.
DIA. (IN.)
JUNCTION NO.
LENGTH/SLOPE (%) (FT)

Figure 8: A Portion of Testville B, 96 Acres

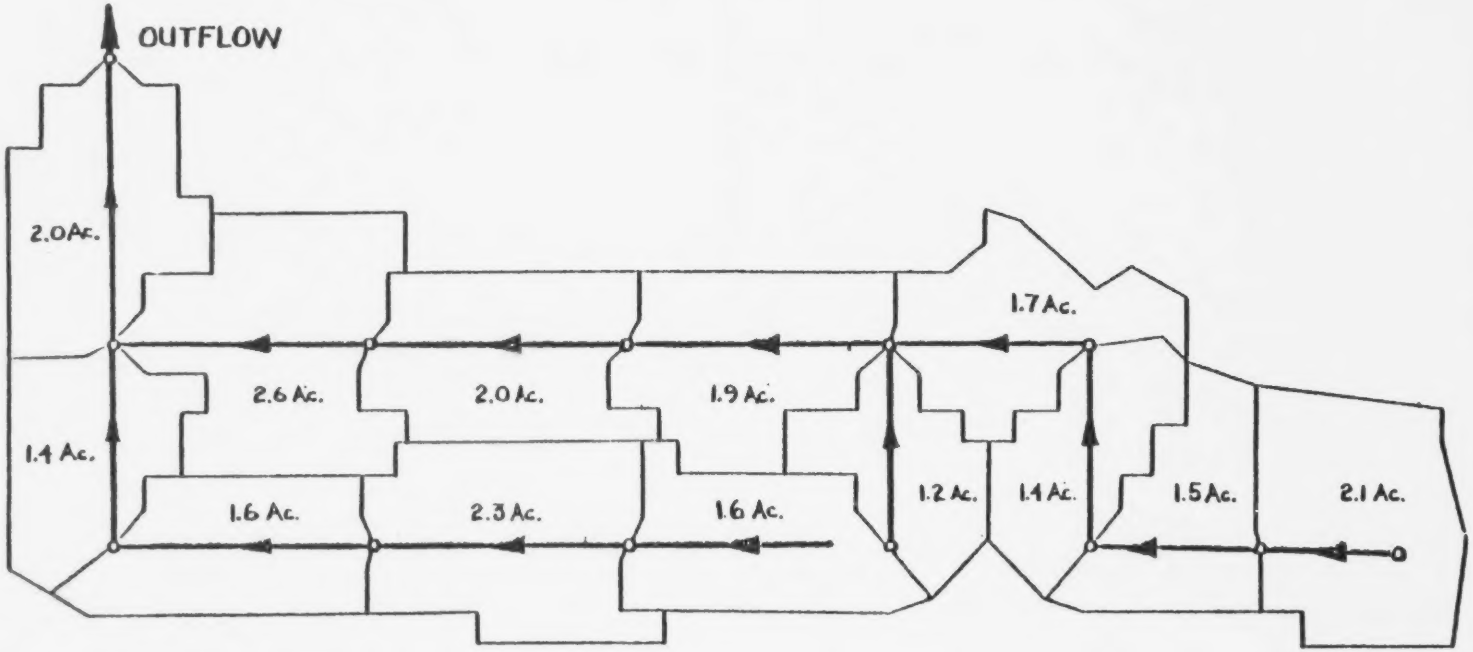

OUTFLOW
2.0Ac.
1.4 Ac.
2.6 Ac.
2.0 Ac.
1.9 Ac.
1.7 Ac.
1.6 Ac.
2.3 Ac.
1.6 Ac.
1.2 Ac.
1.4 Ac.
1.5 Ac.
2.1 Ac.

Figure 9: Schematic of a Typical Small Residential Watershed Used for Systematic Comparison of the RM and SWMM Flows.

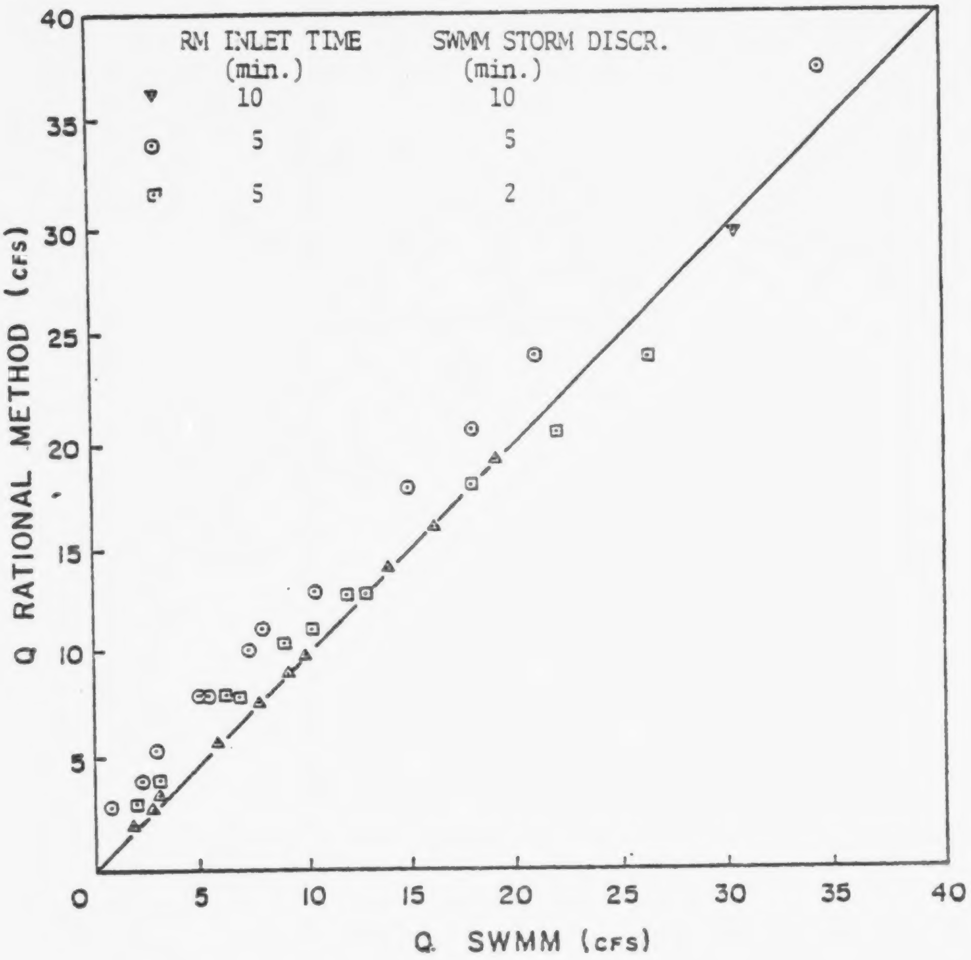


Figure 10: Calibration of Inlet Time to Obtain Flows in Agreement with SWMM.

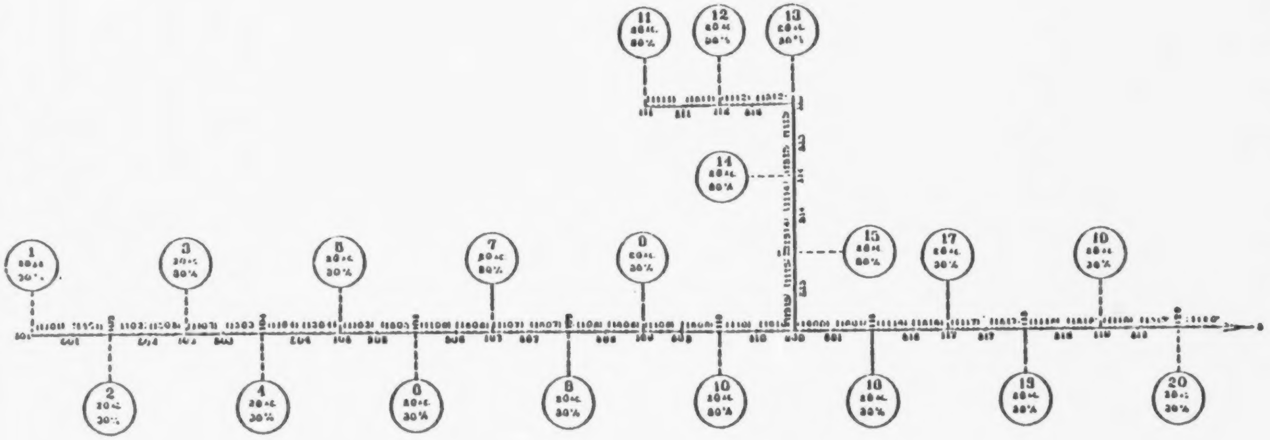

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20

Figure 11: Schematic Layout of Testville A.

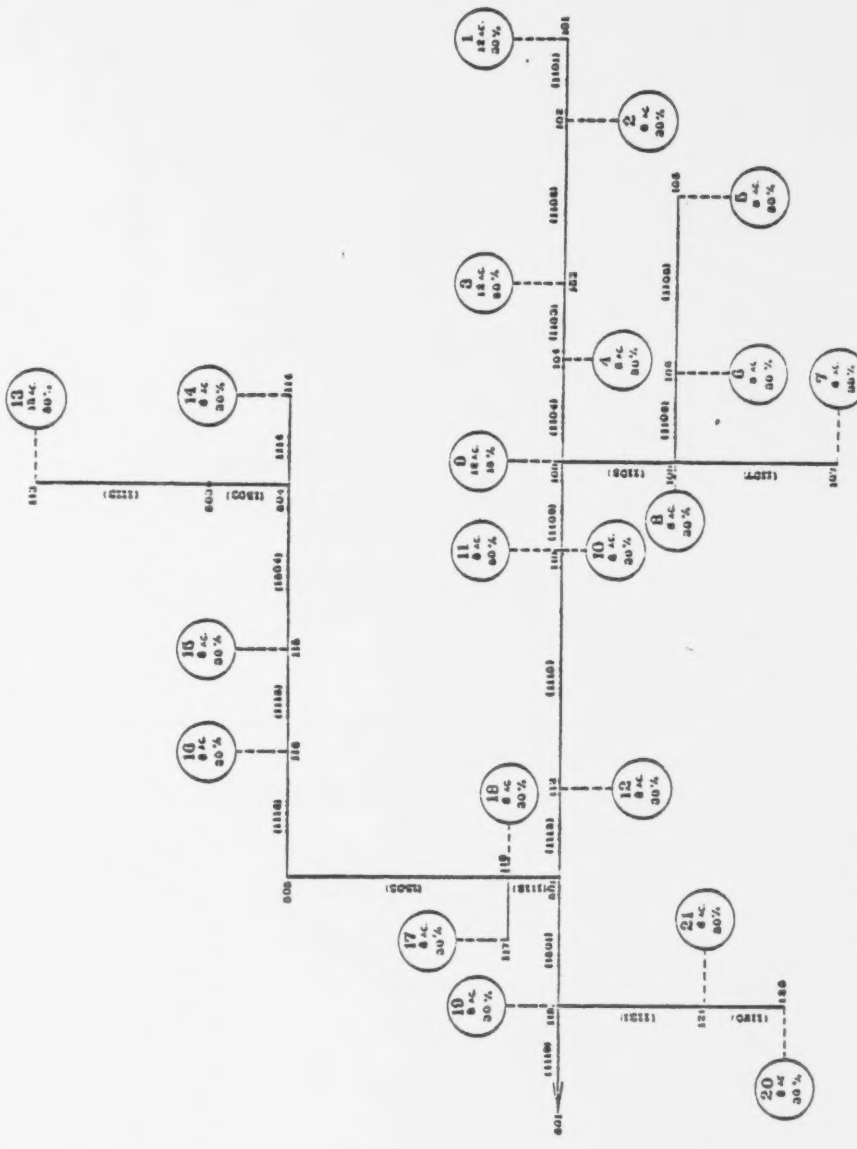


FIGURE 12. Schematic layout of TESTVILLE B.

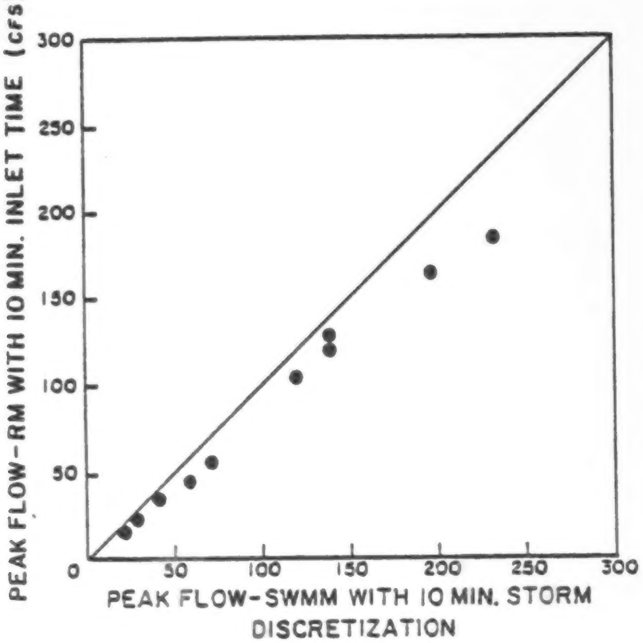


Figure 13: Comparison of Peak Flows Calculated with SWMM and RM for Testville B

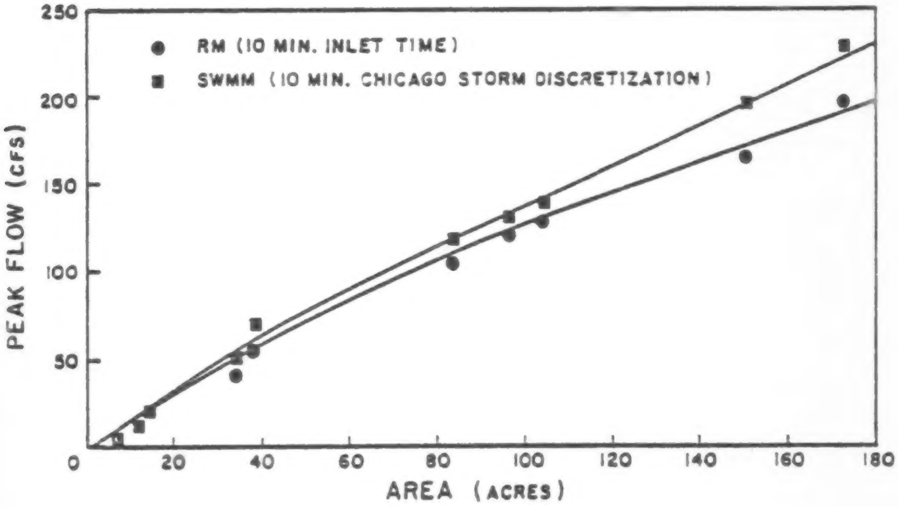


Figure 14: Peak Flows vs. Area for Testville B

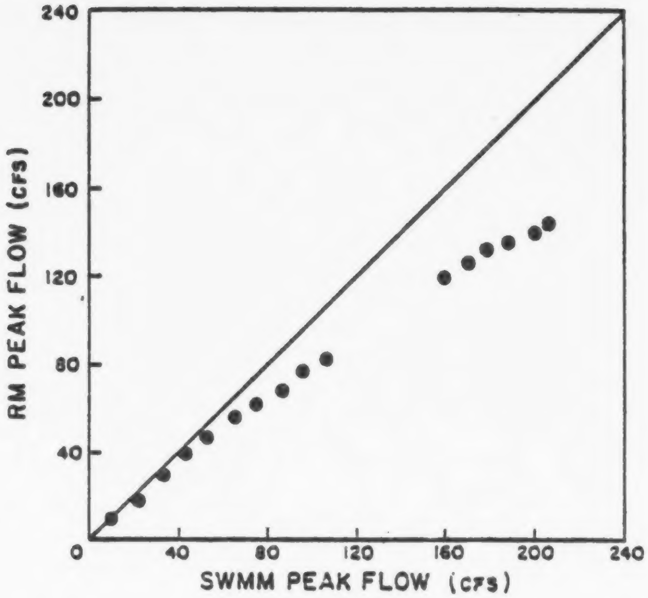


Figure 15: Comparison Between Peak Flows Calculated with SWMM and RM for Testville A.

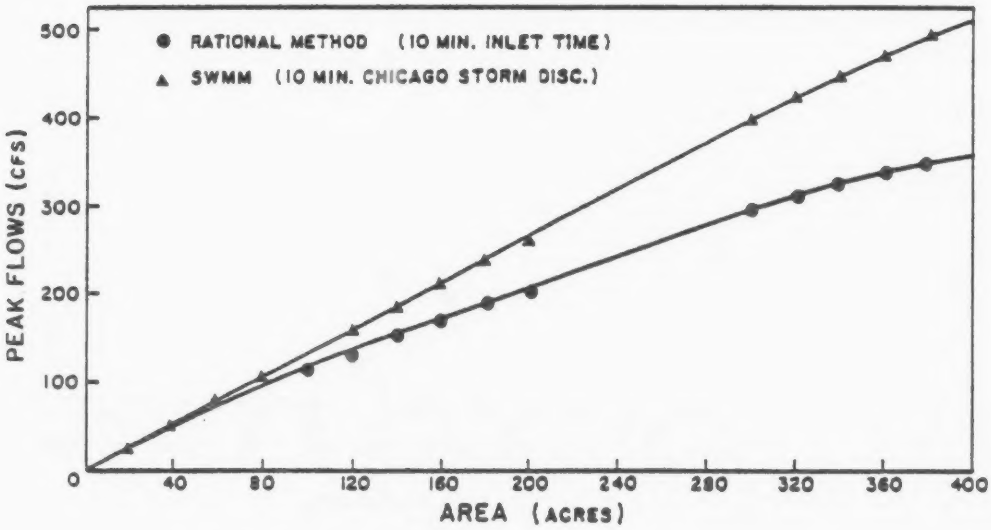


Figure 16: Peak Flows vs. Area for Testville A.

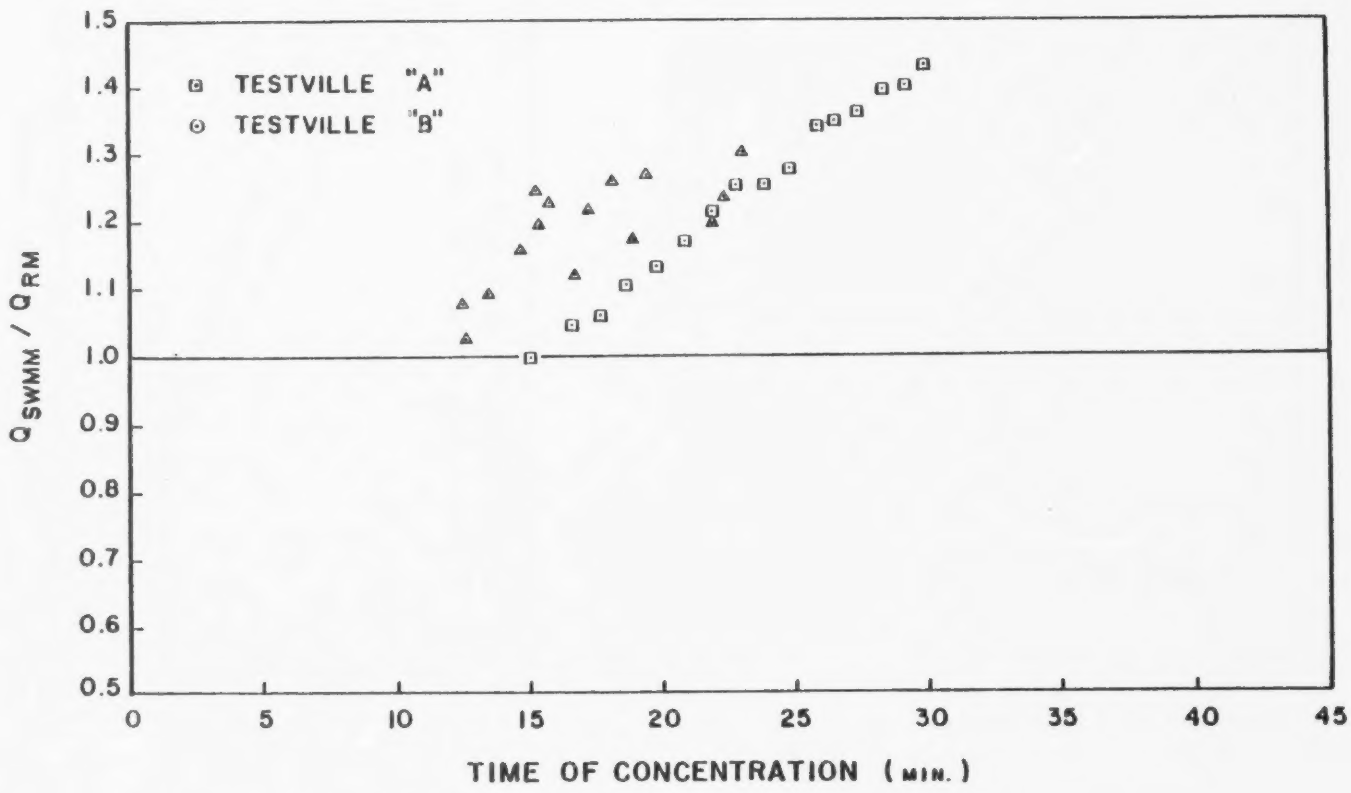


Figure 17: Ratio of Q_{SWMM}/Q_{RM} in Terms of the Time of Concentration.

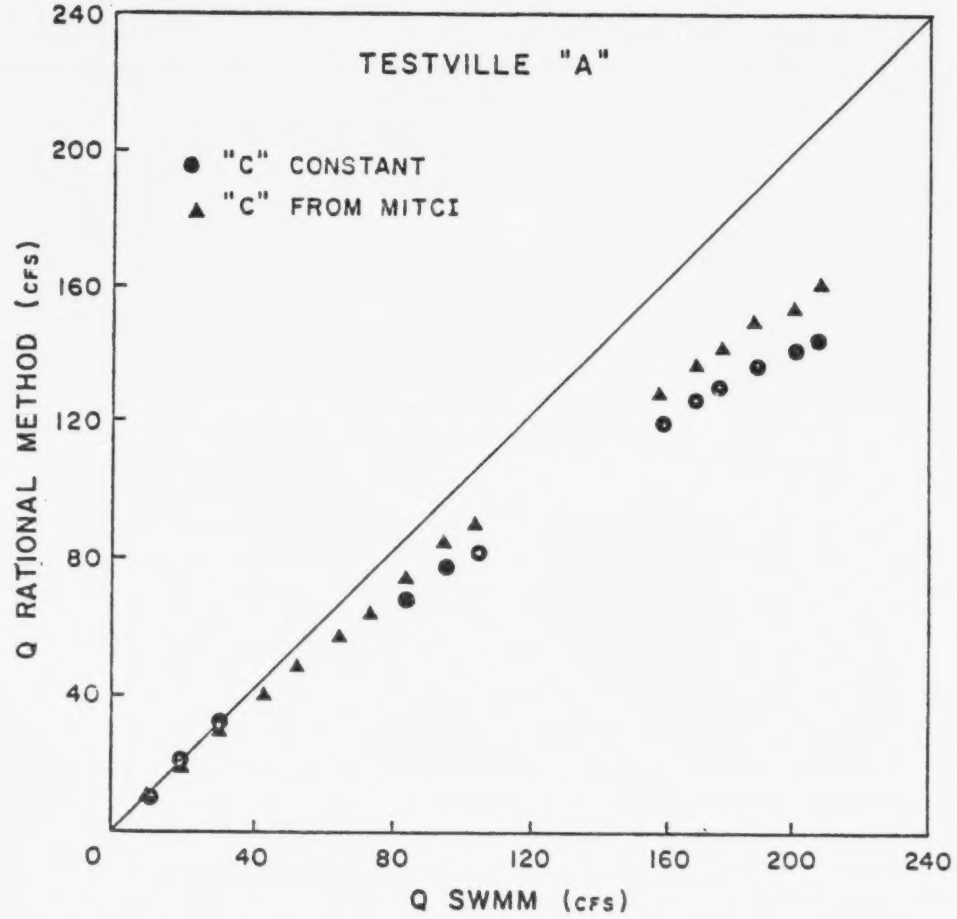


Figure 18: Rational Method Flows with Runoff Coefficient Variable with the Time of Concentration (Mitci (15)) as Compared to Flows with Constant C and SWMM Flows - Testville A.

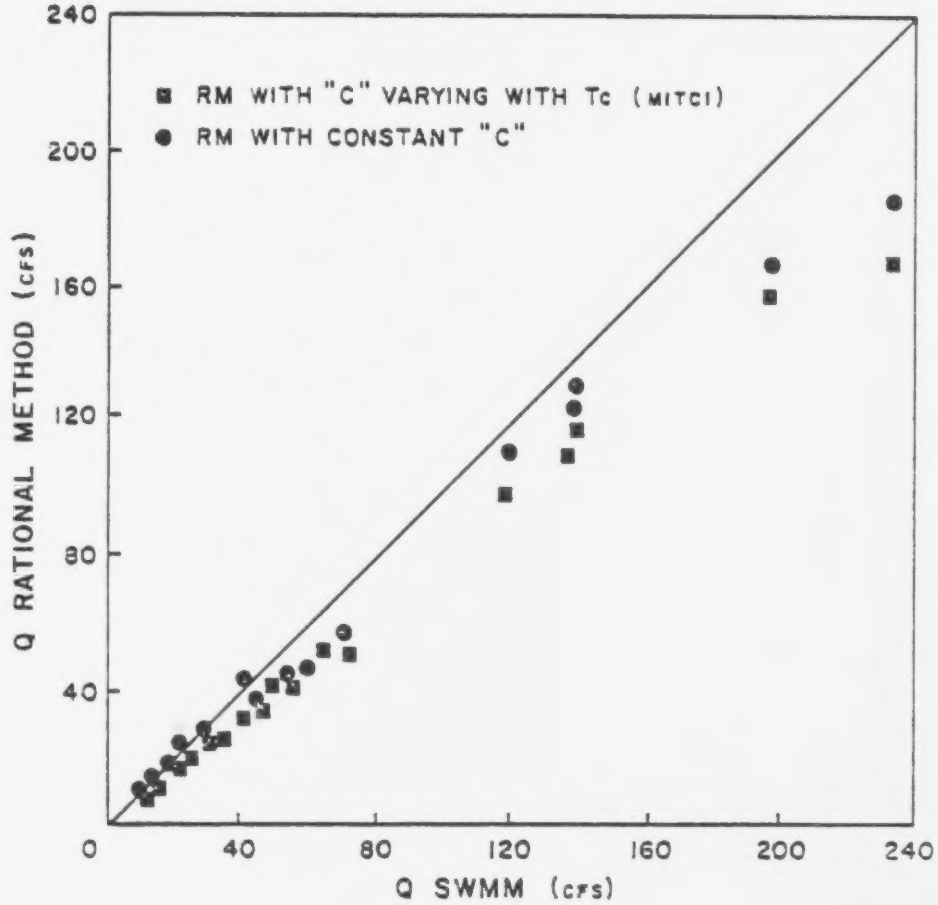


Figure 19: Rational Method Flows with Runoff Coefficient Variable with the Time of Concentration (Mitci (15)) as Compared to Flows with Constant C and SWMM Flows - Testville B.

URBAN RUNOFF QUALITY IN METROPOLITAN TORONTO

by

F. Ivan Lorant,[1] P.Eng

1. BACKGROUND

Following the Agreement between the United States and Canada on Great Lakes Water Quality in 1972, a number of research projects were undertaken under the guidance of Canada-Ontario Urban Drainage Subcommittee to carry out research programs on the abatement of municipal pollution. One of the first studies included in this program was the collection of field data on the quantity and quality of urban runoff in the Borough of East York, Toronto. The study included the setup of a data acquisition system, collection of data on quantity and quality, the calibration and verification of the Storm Water Management Model Runoff and Transport Blocks.

A previous paper presented in 1977 (see reference 1 in the Bibliography) described in detail the data acquisition system and the calibration of the quantity aspects of the Storm Water Management Model. The intent of this paper is to summarize the water quality information observed over the period December 1975 - September 1976.

The total test area of 383 acres shown on Figure 1 was developed in the 1920's and is served by a combined sewer system. The predominant land use of the urban catchment area is residential (89.1%) made up by single family residential units, while the remaining area is institutional (5.7%), parks and open spaces

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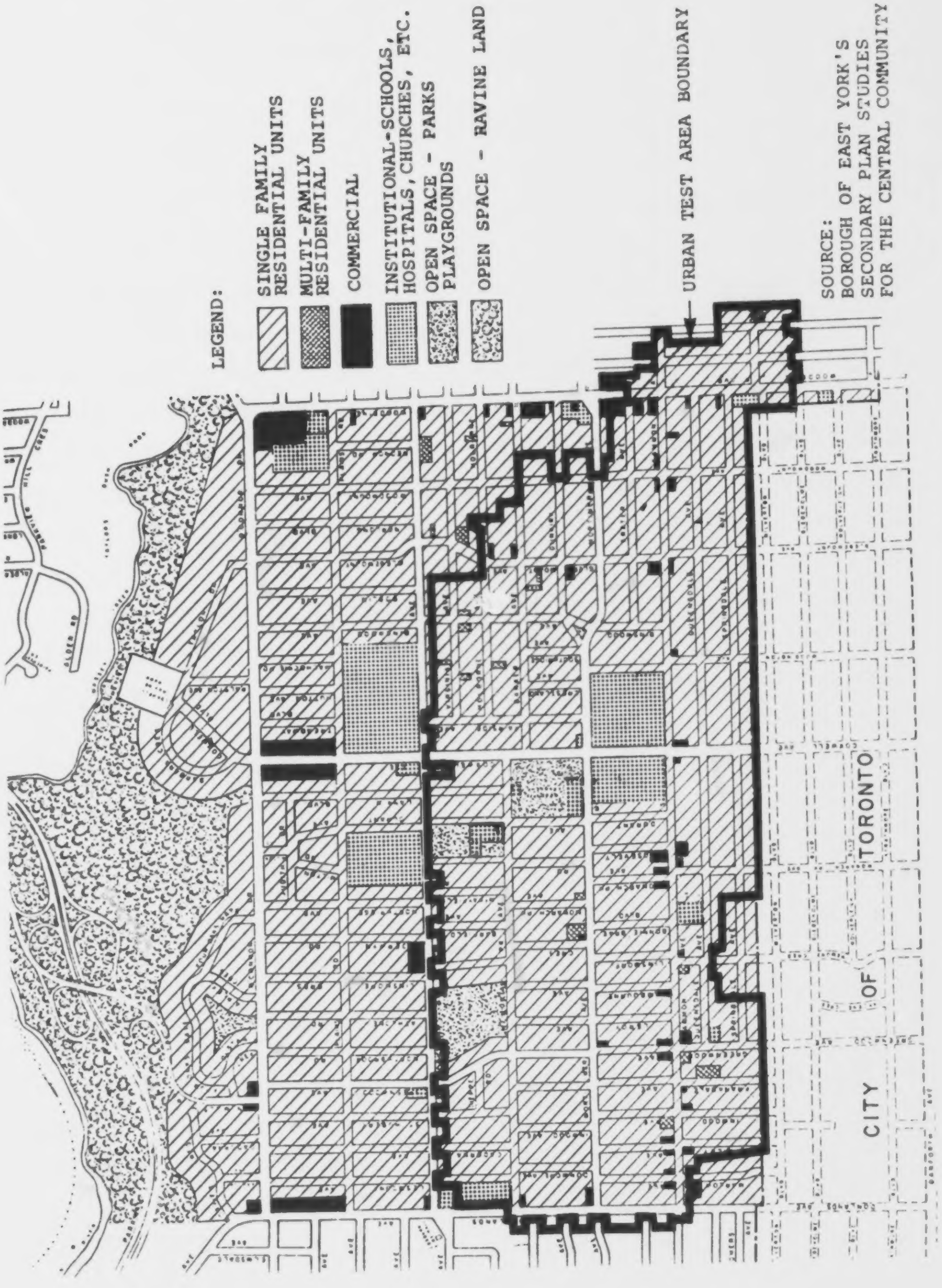
Reproduced from best available copy.


LEGEND:
▨ SINGLE FAMILY RESIDENTIAL UNITS
▩ MULTI-FAMILY RESIDENTIAL UNITS
■ COMMERCIAL
▦ INSTITUTIONAL-SCHOOLS, HOSPITALS, CHURCHES, ETC.
OPEN SPACE - PARKS PLAYGROUNDS
OPEN SPACE - RAVINE LAND
URBAN TEST AREA BOUNDARY
CITY OF TORONTO

SOURCE: BOROUGH OF EAST YORK'S SECONDARY PLAN STUDIES FOR THE CENTRAL COMMUNITY

FIGURE 1

EXISTING LAND USE

(4.2%) and commercial (1%). There is no accurate data available on the population, the estimated population of the area is 14,600, with an average of 3.5 persons per household, and an average gross residential density of 38 persons per acre.

A detailed analysis of photographs and site inspection determined that 49% of the study area i.e. 187 acres can be classified as impervious, made up as follows:

- 52% roads, sidewalks and driveways
- 38% roofs connected to the sewers
- 4% roofs not connected to the sewers; and
- 6% miscellaneous.

A special survey determined that there are 481 catch basins, therefore, the catch basin density is 1.26 per acre. The inspection of the sumps showed that the total storage volume is approximately 13 cu. ft. in each sump.

Limited records indicate that all pipes up to 24 in. in diameter are glazed vitrified clay, while the remaining larger sizes are concrete, except two 60 and 66 in. diameter sewers which are believed to be brick.

2. MUNICIPAL PRACTICES

A number of the municipal practices such as street cleaning, sewer cleaning, snow and ice control can affect the quantity and quality of storm water runoff.

During the snow and ice free period between April and November, the roads within the study area are generally swept by a pick-up sweeper once a week. In adverse weather conditions, the road gutters are cleaned by manual labour. The total quantity of litter and debris removed was 504 cu. yd. during March - November 1976, and 280 cu. yd. during April - October 1977.

There is no regular program for cleaning sewers. Catch basins are generally cleaned once a year by a vacuum hose. No records are available on the debris removed, however, estimates show that in 1976 the total amount of debris removed was approximately 92,000 lbs.

During the winter period rock salt is used with a 6 ton salt spreader, adding occasionally anti-skid material of sand mixed with the rock salt. The rate of application depends on the weather conditions. During the December 1975 - March 1976 winter, the amount of salt spread varied from 0 to 24 tons per day, totalling 397 tons.

Road plowing begins when the snow accumulation reaches about 3 in., however sidewalks are plowed generally after a snowfall of 1½ in. The snow from the roads and sidewalks are plowed into the road gutter and the snow is only removed if the accumulation along the road becomes unmanageable.

3. DATA ACQUISITION SYSTEM

The purpose of the data acquisition system was to provide accurate and sufficient information in order to prepare precipitation hyetogrpahs, air temperature graphs, flow hydrographs and pollutographs. This data collected was used for calibration and verification of the Storm Water Management Model.

The data acquisition system shown on Figure 2 included the following:

Precipitation Measurement

- A tipping bucket rain gauge with a print-out recorder which prints the actual date and time to the nearest minute every time the precipitation gauge bucket tips, indicating 0.01 of an inch of rainfall. This gives print-out capability up to 36 in. per hour rainfall intensity.

Flow Measurement

- The flow measuring device was installed in a 66 in. diameter outlet pipe and it consisted of a partial trapezoidal weir which was calibrated by model tests.

- A bubbler type system measured the head of water and recorded it on a two pen strip chart recorder together with the air temperature measurement.

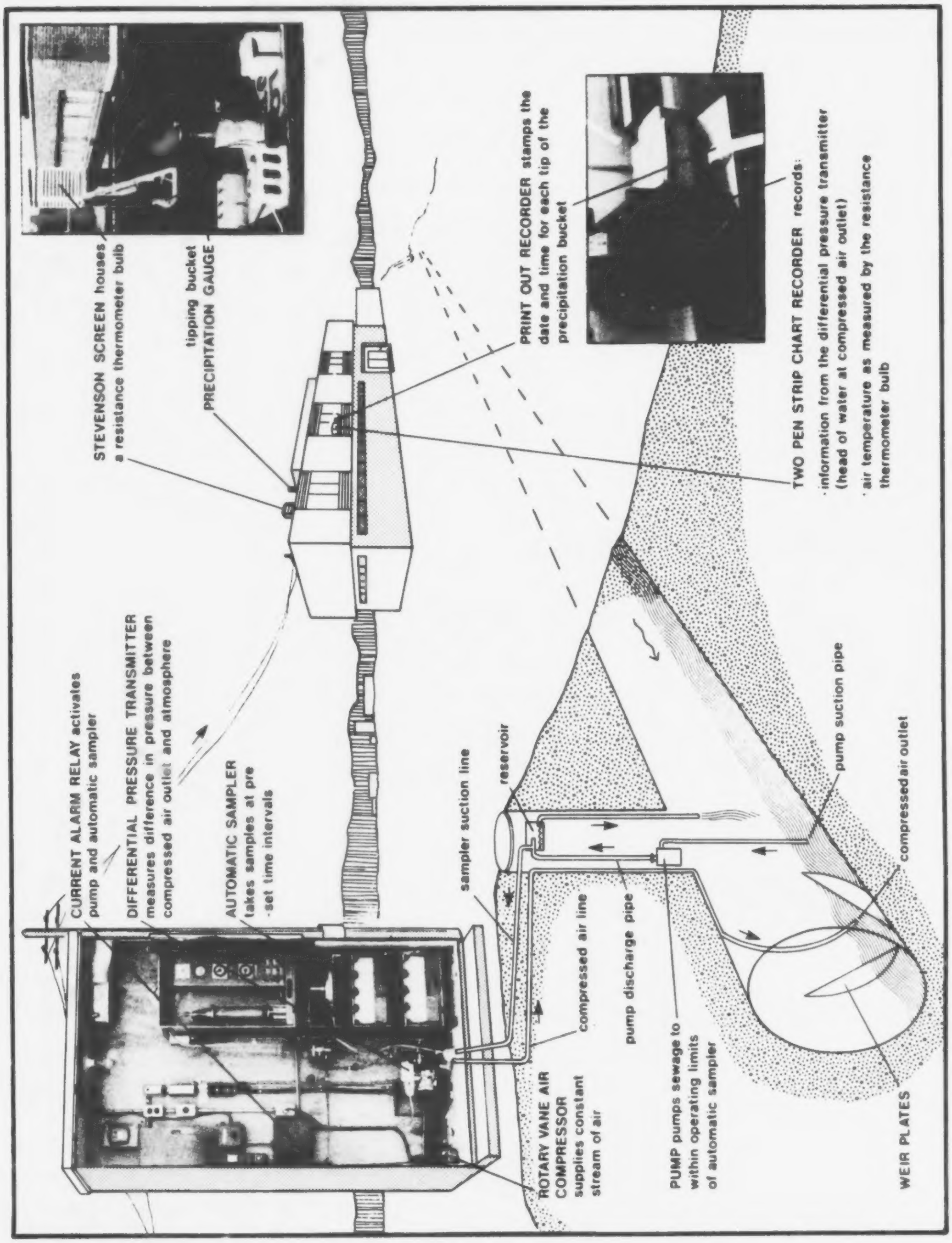

STEVENSON SCREEN houses a resistance thermometer bulb
tipping bucket PRECIPITATION GAUGE
PRINT OUT RECORDER stamps the date and time for each tip of the precipitation bucket
TWO PEN STRIP CHART RECORDER records:
· information from the differential pressure transmitter (head of water at compressed air outlet)
· air temperature as measured by the resistance thermometer bulb
CURRENT ALARM RELAY activates pump and automatic sampler
DIFFERENTIAL PRESSURE TRANSMITTER measures difference in pressure between compressed air outlet and atmosphere
AUTOMATIC SAMPLER takes samples at pre-set time intervals
sampler suction line
reservoir
pump suction pipe
compressed air outlet
ROTARY VANE AIR COMPRESSOR supplies constant stream of air
compressed air line
pump discharge pipe
PUMP pumps sewage to within operating limits of automatic sampler
WEIR PLATES

FIGURE 2
SCHEMATIC OF DATA ACQUISITION SYSTEM

Water Quality Sampler

- The automatic sampler capable of collecting 24 discrete 1,000 ml. samples was located above ground. This arrangement in spite of the 38 foot head difference between the sample jars and the sewer, was the most appropriate, as regulations would have required to explosion proof all electric instrumentation in the manhole.

- The water quality samples of dry and wet weather flows, were analysed at the Ontario Ministry of the Environment laboratories.

Air temperature measurements were taken by a resistance thermometer bulb. Recording of the temperature was on a strip chart recorder.

Time synchronization was achieved by recording all events except the precipitation on a two strip chart recorder and all recorders kept excellent time throughout the study.

4. DRY WEATHER FLOWS

The average measured dry weather flow was 2.75 cfs. Daily and hourly variations were insignificant, when compared to the wet weather flows.

The two pollutants analysed were BOD, average 95.3 mg/l, and suspended solids, average 96.9 mg/l. A detailed summary of the daily and hourly dry weather flow and pollutant variations are shown in Table 1.

5. MODELLING OF FLOWS

The SWMM computer program used in the study was based on the U.S. - E.P.A. Release II, dated September 1970, updated February 1975, with snowmelt program added in October 1975. The quantity calibration was based on low intensity precipitation events, therefore, none of the recorded events had sufficiently high intensities to overcome the estimated infiltration and depression storage losses of pervious areas. Three events have been used to calibrate the model. Subsequently, 20 precipitation - runoff events were used to verify the model. The results were encouraging as shown by the following summary:

RUNOFF VERIFICATION
PER CENT OF CALCULATED DATA

	ERROR		
Calculated Data	<20%	<10%	<5%
Flow Volumes	100%	81%	44%
Peak Flow Rates	91%	59%	34%
Time to Peak	97%	88%	72%

TABLE 1
DAILY AND HOURLY DRY WEATHER FLOW VARIATIONS

	RATIO OF AVERAGE		
DAY	Dry Weather Flow (Average 2.75 cfs)	BOD of Dry Weather Flow (Average 95.3 mg/l)	SS of Dry Weather Flow (Average 96.9 mg/l)
1 - Sunday	0.942	1.000	1.000
2 - Monday	0.985	1.000	1.000
3 - Tuesday	1.036	1.000	1.000
4 - Wednesday	1.076	1.000	1.000
5 - Thursday	1.033	1.000	1.000
6 - Friday	0.975	1.000	1.000
7 - Saturday	0.945	1.000	1.000
HOUR			
1	0.806	0.826	0.766
2	0.710	0.330	0.472
3	0.660	0.175	0.273
4	0.631	0.206	0.231
5	0.617	0.165	0.199
6	0.639	0.144	0.178
7	0.763	0.372	0.818
8	0.947	1.321	1.784
9	1.129	1.672	1.689
10	1.194	1.548	1.448
11	1.224	1.662	1.448
12	1.204	1.362	1.542
13	1.191	1.176	1.427
14	1.205	1.011	1.175
15	1.188	1.053	0.913
16	1.130	1.094	0.787
17	1.101	0.970	0.735
18	1.134	1.104	1.007
19	1.184	1.620	1.689
20	1.165	1.775	1.511
21	1.093	1.465	1.280
22	1.064	1.053	1.049
23	1.048	0.888	0.766
24	0.971	1.001	0.797

6. WATER QUALITY DATA

The runoff quality data analysed are listed in Table 2 with approximate range of values.

TABLE 2
RANGE OF OBSERVED WATER QUALITY DATA

Pollutants	Range	Remarks
BOD	25-250 mg/l	High initial values
COD	100-1,000 mg/l	High initial values
Chloride Cl	30-2,000 mg/l	Higher during winter
Sulphate SO_4	0.5-70 mg/l	Higher during winter
Conductivity UMHOS/cm	200-7,000	Higher during winter
Lead Pb	0.2-3 mg/l	Higher during winter
Total phosphorus P	0.5-5 mg/l	
Dissolved reactive phosphorus P	0.1-1.0 mg/l	
Phenols	3-31 part per billion	Higher during winter
Free ammonia	0.2-7 mg/l	
Total Kjeldahl	4-30 mg/l	
Nitrite	0.1-0.7 mg/l	
Nitrate	0.1-2.0 mg/l	
Suspended solids	(dried or ashed) 30-2,000 mg/l	
Dissolved solids	(dried or ashed) 100-4,000 mg/l	
Total solids	(dried or ashed) 100-4,000 mg/l	

A sample sheet of the recorded data is shown in Table 3.

Due to the shortage of funds and time constraints the project included only limited amount of calibration and verification of the BOD and suspended solid pollutographs. However, in order to permit future calibration and verification studies of any of the pollutants, all recorded data will be published in early 1980. (Reference 2).

7. MODELLING OF WATER QUALITY

The automatic sampler was programmed to record and collect samples from significant storm events covering a reasonable time span. This was achieved by activating the sampler when the flow reached 15 cfs and continued to take samples at 6 minute 40 second intervals. Unfortunately this meant that the initial part of the recorded pollutograph was not available.

Before carrying out any calibration of the quality parameters simulated hydrographs were adjusted to agree with recorded hydrographs.

Biochemical Oxygen Demand (BOD)

The observed BOD values generally start in the order of 150 mg/l but after one to two hours the readings drop to 25 mg/l. Out of the 18 events monitored only two had initial BOD values higher than 200 mg/l.

BOD values can drop rapidly, however, the simulation could not duplicate this rapid drop. Generally the initial stage of simulations are below the observed values, while the latter part of the simulated pollutographs are higher than the

TABLE 3

RUNOFF QUALITY DATA

Units are mg./litre unless otherwise indicated

DATE DY/MO/YR	TIME HR MN	FLOW c.f.s	B.O.D	C.O.D	CHLORIDE AS Cl	SULPHATE AS SO_4	CONDUCTIVITY UMHOS/CM	LEAD AS Pb	TOTAL PHOSPHORUS AS P	DISSOLVED REACTIVE PHOSPHORUS AS P	PHENOLS IN PPB *	NITROGEN AS N				SUSPENDED SOLIDS			DISSOLVED SOLIDS			TOTAL SOLIDS		
												FREE AMMONIA	TOTAL KJELDAHL	NITRITE	NITRATE	DRIED	ASHED	LOSS	DRIED	ASHED	LOSS	DRIED	ASHED	LOSS
30/12/75	14:46	19.2	61	255	747	37	2600	0.51	2.70	1.00	24	1.8	9.1	0.04	0.4	180	90	90	1480	1390	90	1660	1480	180
	14:52	20.8	65	495	718	37	2600	0.48	2.10	0.74	26	2.1	8.7	0.04	0.4	140	70	70	1440	1370	70	1580	1440	140
	14:59	20.8	75	260	687	34	2500	0.53	2.60	0.60	23	1.3	10.0	0.02	0.2	160	80	80	1470	1400	70	1630	1480	150
	15:06	20.0	64	250	683	33	2500	0.49	1.70	0.44	23	1.9	7.2	0.04	0.2	120	65	55	1380	1320	60	1500	1385	115
	15:12	19.0	43	260	697	33	2500	0.55	1.00	0.34	24	1.7	3.3	0.08	0.3	120	70	50	1440	1370	70	1560	1440	120
	15:19	18.8	45	160	712	33	2500	0.53	0.76	0.22	23	1.9	5.4	0.10	0.3	110	65	45	Exh.	Exh.	Exh.	Exh.	Exh.	Exh.
	15:26	16.7	38	185	790	36	2900	0.53	1.20	0.36	24	3.3	6.8	0.08	0.3	120	70	50	1650	1610	40	1770	1680	90
	15:32	15.8	33	140	810	37	2950	0.46	1.00	0.20	25	2.6	5.6	0.28	0.3	105	70	35	1670	1620	50	1775	1690	85
	15:39	15.0	33	145	881	37	3150	0.43	0.84	0.14	26	1.8	4.7	0.34	0.5	100	60	40	Exh.	Exh.	Exh.	Exh.	Exh.	Exh.
	15.46	14.2	35	140	916	38	3300	0.54	0.44	0.18	27	2.7	3.9	0.34	0.7	60	20	40	1760	1690	70	1820	1710	110
	15:52	13.3	31	185	930	40	3300	0.19	0.76	0.34	25	3.0	4.5	0.34	0.7	160	100	60	Exh.	Exh.	Exh.	Exh.	Exh.	Exh.
	15:59	12.8	31	150	971	40	3400	0.32	0.68	0.20	25	3.1	5.8	0.30	0.5	140	80	60	1850	1760	90	1990	1840	150
	16:06	12.3	31	145	975	53	3550	0.44	0.84	0.18	24	2.7	5.6	0.44	0.8	110	60	50	Exh.	Exh.	Exh.	Exh.	Exh.	Exh.
	16:12	11.5	38	160	1072	46	3600	0.54	0.96	0.30	26	3.6	6.2	0.40	1.0	150	90	60	Exh.	Exh.	Exh.	Exh.	Exh.	Exh.
	16:19	10.7	35	235	1051	44	3800	0.57	1.00	0.42	25	4.3	6.6	0.46	0.5	140	80	60	2070	2000	70	2210	2080	130
	16:26	10.6	58	160	1025	46	3700	0.42	0.80	0.26	24	2.7	5.4	0.02	0.2	140	70	70	2070	1980	90	2210	2050	160
	16:32	10.6	53	160	1034	48	3800	0.26	1.00	0.30	28	3.9	7.2	0.16	0.2	100	60	40	1930	1840	90	2030	1900	130
	16:39	10.4	48	190	1091	55	4000	0.26	0.96	0.38	27	4.3	7.0	0.08	0.2	140	70	70	2070	1990	80	2210	2060	150
	16:46	10.0	58	245	1072	50	3900	0.27	1.20	0.34	28	4.1	7.4	0.04	0.2	140	80	60	2010	1940	70	2150	2020	130
	16:52	9.8	46	185	1080	50	3900	0.21	1.10	0.34	27	3.9	7.2	0.66	0.7	130	70	60	1960	1890	70	2090	1960	130
	16:59	9.7	44	155	1033	49	3300	0.21	1.00	0.32	25	4.9	8.0	0.36	0.4	130	70	60	1960	1880	80	2090	1950	140
	17:06	9.5	45	210	1007	49	3750	0.22	1.20	0.30	23	4.6	4.3	0.02	0.2	160	70	90	1890	1820	70	2050	1890	160

* Samples Unpreserved

observed values. During calibration varying the BOD concentration of the catch basins had little effect on the accuracy. Examples of simulated and observed BOD pollutographs are shown on Figures 3 and 4.

Suspended Solids

For the simulation of suspended solid loads the empirical method (1SS=1) was selected as it produced better results than the experimental equation (1SS=0). In the computation of dust and dirt accumulation, observed dates of street cleaning activities were utilized. The following equation was developed to estimate the number of equivalent days of pollution accumulation.

$$N_D = t_1 \; (1-E)^{n-1} + t_2 \; (1-E)^{n-2} + \ldots \; t_n \; (1-E)^{n-n}$$

To estimate the length of dry period, the time of the last storm was selected when the total rainfall amounted to one inch. Time intervals between street sweeping and storm events in days are represented by t. E is the efficiency of the street sweeping and n represents the number of time intervals.

An illustration of the above system is shown in the following table for the 31 July 1976 storm event.

Storm analysed = 31 July 1976
Last storm (minimum one inch) = 20 July 1976
Street sweeping events during the two storms = 21 and 27 July
Therefore, t_1 = 21 July - 20 July = 1 day
t_2 = 27 July - 21 July = 6 days
t_3 = 31 July - 27 July = 4 days
n = number of events = 3
E = street sweeping efficiency = 0.75

$$N_D = 1(1-0.75)^2 + 6(1-0.75)^1 + 4(1-0.75)^0 = 5.56 = 6 \text{ days}$$

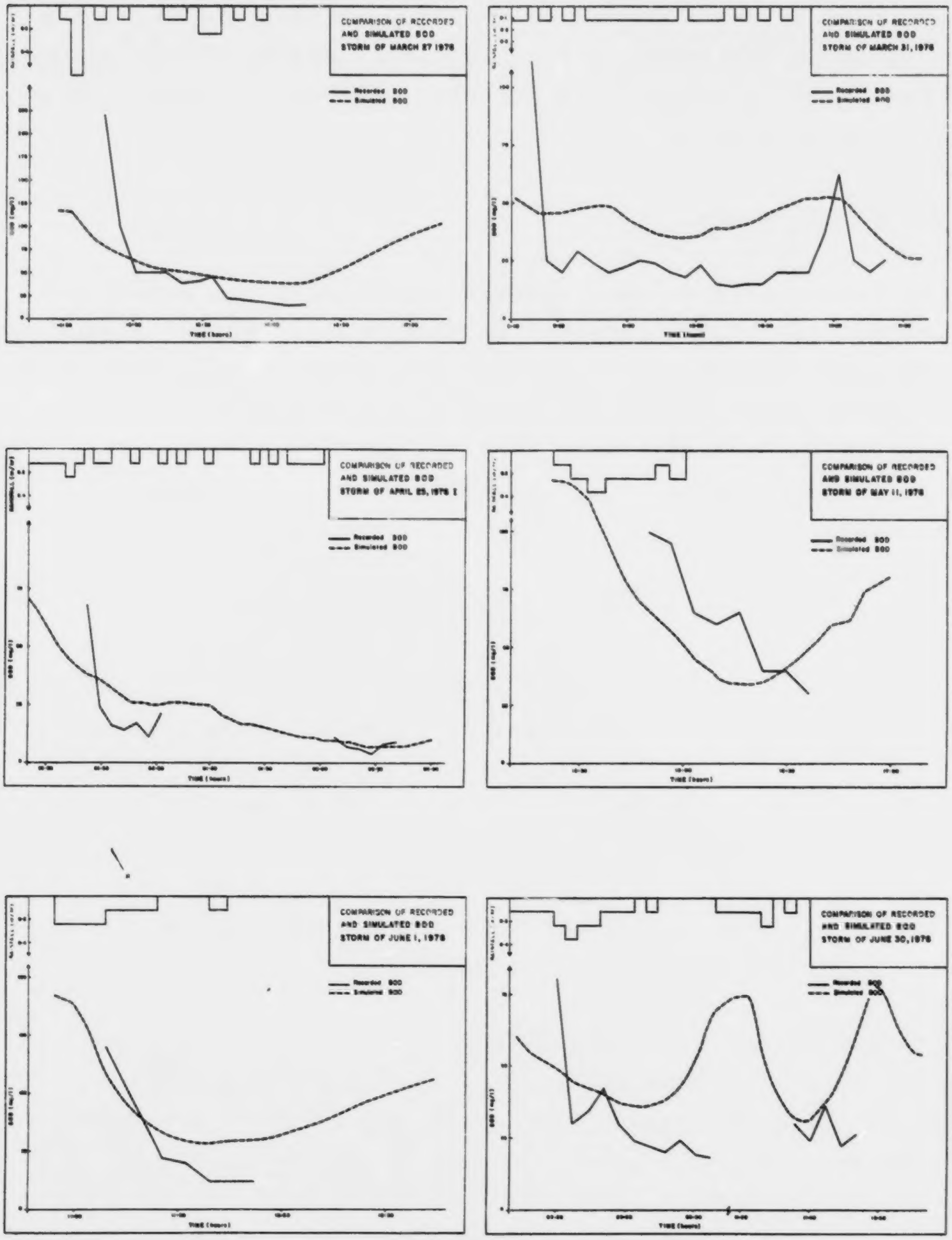


FIGURE 3
COMPARISON OF RECORDED AND SIMULATED B O D

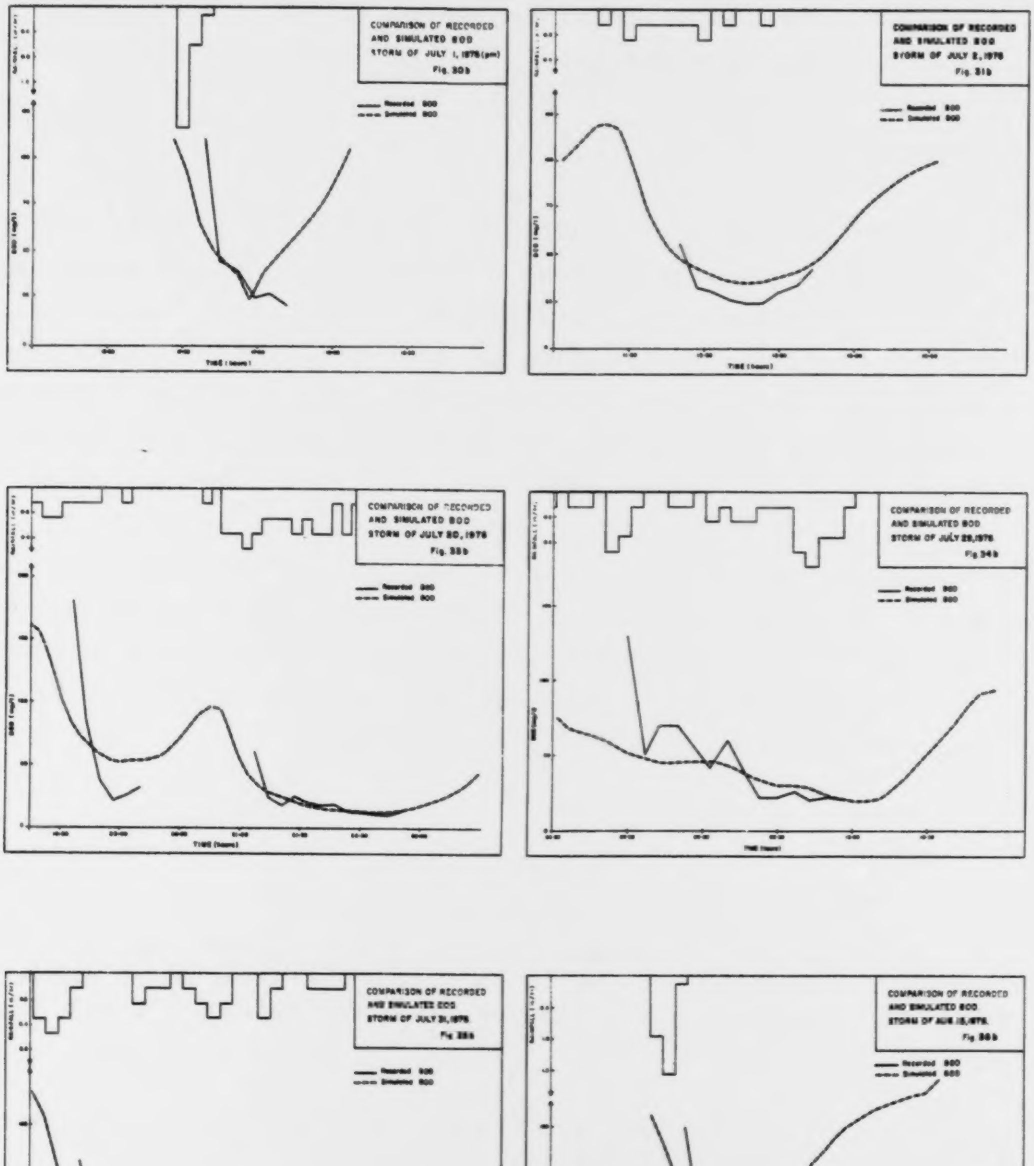


FIGURE 4
COMPARISON OF RECORDED AND SIMULATED B O D

The simulated and observed suspended solids pollutographs are shown on Figures 5 and 6.

Conclusions

The verification of the BOD and suspended solid pollutographs demonstrated the possibility of simulating urban runoff water quality parameters by a calibrated Storm Water Management Model. Although the results of the verification exercise were promising, the accuracy of predicting BOD or SS pollutographs cannot reach the accuracy attained in simulating flow hydrographs by a calibrated SWMM. However, both the Suspended Solid and BOD simulated pollutographs presented could be improved after an extensive re-calibration program. It is hoped that the publication of the entire observed quantity and quality data will provide the necessary impetus and incentive for research students or other non-academic modellers to improve the capabilities of the SWM model.

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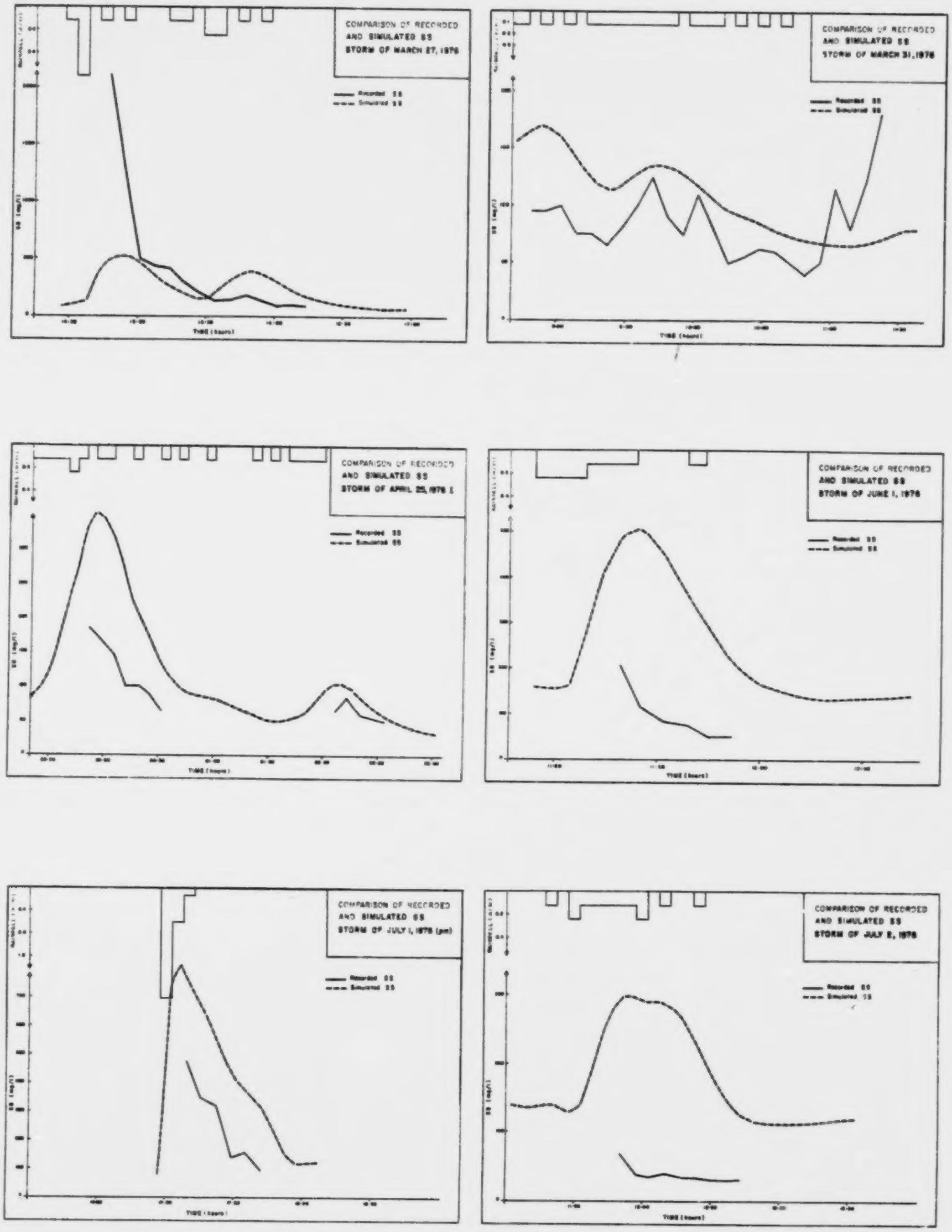

FIGURE 5
COMPARISON OF RECORDED AND SIMULATED SUSPENDED SOLIDS

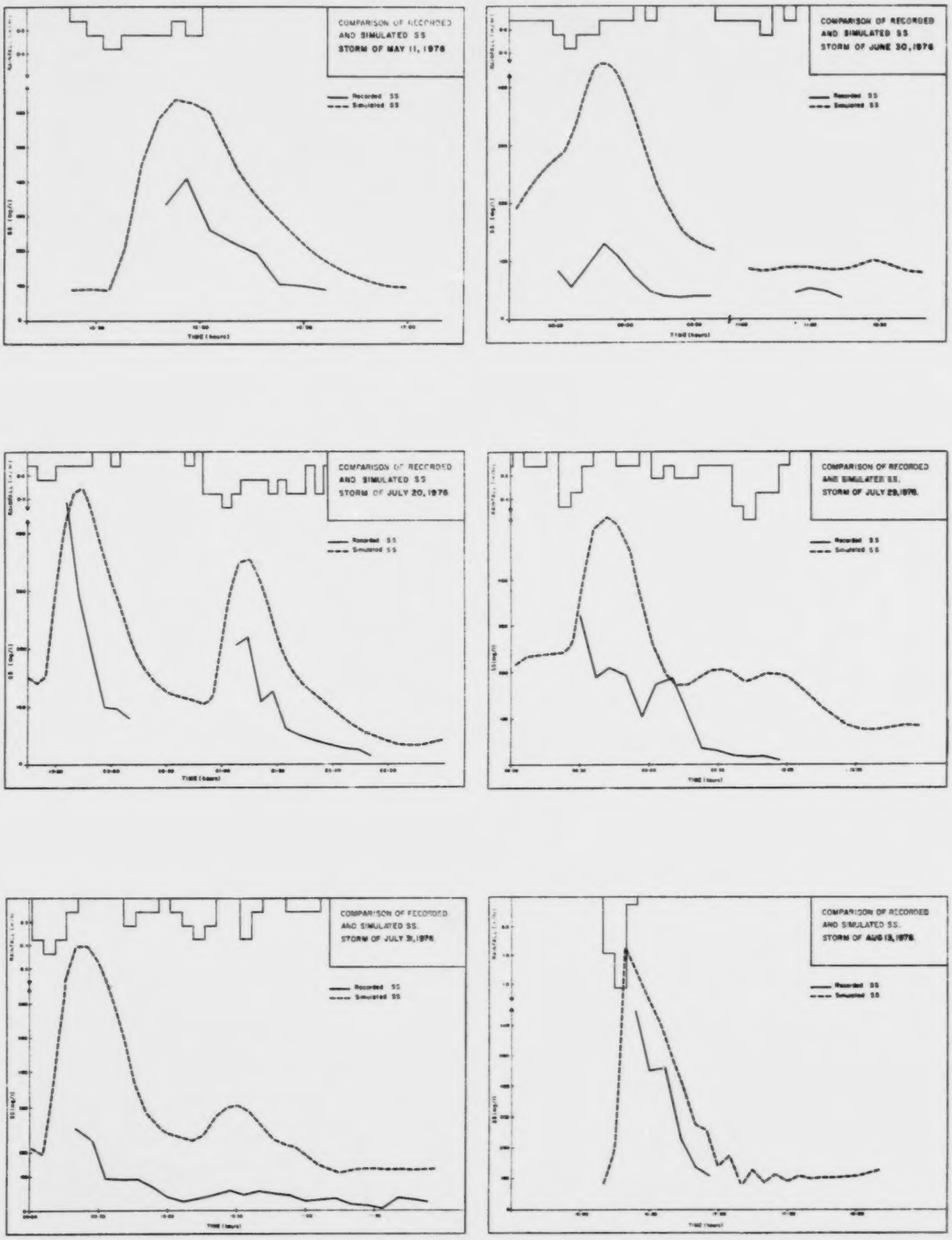

FIGURE 6
COMPARISON OF RECORDED AND SIMULATED SUSPENDED SOLIDS

AN EXAMINATION OF THE STORM WATER MANAGEMENT MODEL (SWMM) SURFACE-RUNOFF-QUALITY ALGORITHMS

By William M. Alley[1]

INTRODUCTION

The Storm Water Management Model (SWMM) is a computer model for simulation of storm- and combined-sewer systems (Metcalf and Eddy, Inc., and others, 1971). The surface-runoff-quality algorithms contained in SWMM have formed the basis for similar algorithms contained in several other urban-runoff models including STORM (U.S. Army Corps of Engineers, 1976) and a modified ILLUDAS model (Han and Delleur, 1979). These models, in some instances, have been used extensively. For example, STORM (Storage, Treatment, Overflow, and Runoff Model) was the most commonly used model in studies conducted under Section 208 of the Federal Water Pollution Control Act of 1972 (P. E. Shelley and E. D. Driscoll, written commun., 1979).

The purpose of this paper is to examine the functional relationships used in SWMM for surface-runoff-quality simulation and to consider some areas for improvement.

SWMM SURFACE-RUNOFF-QUALITY ALGORITHMS

There are two main components of the surface-runoff-quality algorithms contained in SWMM: Constituent accumulation and constituent washoff. Constituent accumulation is estimated as a function of land use, number of

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days prior to the storm during which the accumulated rainfall was less than 1.0 inch, and street-cleaning frequency and efficiency. The quantity P in milligrams of a constituent I on areas of land use L at the beginning of a storm is computed as:

$$P(I,L) = F(I) \times DD(L) \times DRYDAYS \times GLEN(L) \times 453.6 \qquad (1)$$

where

F(I) = milligram of constituent I per gram of "dust and dirt;"

DD(L) = "dust and dirt" accumulation rate for land use L, in pounds per day per foot of curb;

DRYDAYS = number of days prior to storm during which the accumulated rainfall was less than 1.0 inch (modified to account for street sweeping);

GLEN(L) = curb length for land use L, in feet; and

453.6 = conversion factor (grams per pound)

Constituent washoff is simulated using an exponential decay equation:

$$POFF(I,L) = PO(I,L)\ [1-e^{(-\alpha R \Delta t)}] \qquad (2)$$

where

POFF(I,L) = amount of constituent I removed from areas of land use L during a time step, in milligrams;

PO(I,L) = amount of constituent I on areas of land use L at the beginning of time step, in milligrams;

α = decay coefficient;

R = runoff rate, in inches per hour; and

Δt = time step, in hours

The primary assumption for use of equation 2 is that the amount of constituent washed off during each time step is proportional to the amount remaining on the land surface at the start of the time step.

When equation 2 is used to simulate suspended solids, it is multiplied by an availability factor (ASUS) where

$$\mathrm{ASUS} = 0.057 + 1.4R^{1.1} \qquad (3)$$

Similarly, when computing the washoff of settleable solids, equation 2 is multiplied by an availability factor (ASET) where

$$\mathrm{ASET} = 0.028 + R^{1.8} \qquad (4)$$

Both availability factors are limited to a maximum value of 1.0, and the user has the option of setting either or both of them to 1.0 for all time steps.

After computation of constituent washoff during a time step using equations 2-4, SWMM increases the amount of 5-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total nitrogen, and total phosphate washed off using the relationship:

$$\mathrm{POFF}(I,L)_A = \mathrm{POFF}(I,L)_U + \mathrm{F1}(I) \times \mathrm{SUS} + \mathrm{F2}(I) \times \mathrm{SET} \qquad (5)$$

where F1(I) and F2(I) are correction factors for constituent I; SUS and SET are the amounts of suspended and settleable solids washed off, in milligrams; and the subscripts A and U denote adjusted and unadjusted values, respectively. Equation 5 is utilized to account for the insoluble portion of constituents associated with suspended solids and settleable solids.

AREAS FOR IMPROVEMENT

Modeling experience and data collected since the original development of the runoff-quality portion of SWMM have provided indications of potential areas for improvement of the model. These areas for improvement will be discussed under the general headings of constituent accumulation, constituent washoff, and source identification.

Constituent Accumulation

Several features of the constituent accumulation equation (equation 1) are worthy of note. First, the equation assumes a linear buildup of constituents on the land surface with no upper limit on the amount accumulated. Second, antecedent conditions are defined by the rather arbitrary 1.0-inch DRYDAYS criterion. Third, the assumption is made that the land surface is completely void of constituents at the start of the DRYDAYS period.

Several studies have suggested that the rate of accumulation of constituents on urban surfaces is not linear and that there is a limit to the amount of constituents that can accumulate between storms, regardless of the length of dry period (Sartor and Boyd, 1972; Jewell and others, 1978; and Smith and Jennings, 1979). Data collected by Sartor and Boyd (1972) suggest that the accumulation rate is largest for several days after a period of street cleaning or rainfall, and then the rate decreases and approaches zero. Apparently, constituents are resuspended by wind and land-use activities such as vehicles moving along a highway. Barkdoll, Overton, and Betson (1977) have pointed out that a particle with a diameter of 246 microns could be resuspended by air masses with velocities of less than 5 miles per hour. Sartor and Boyd (1972) found that from 40 to 90 percent of constituent loads on street surfaces are associated with particle sizes less than 246 microns.

Overton and Meadows (1976) suggest the following alternative to equation 1:

$$\frac{dP}{dt} = K_1 - K_2P \qquad (6)$$

where

P = amount of a particular constituent on subareas of a given land use, in pounds per acre;

K_1 = a constant rate of constituent deposition, in pounds per acre per day;

K_2 = a rate constant for constituent removal, in day^{-1}; and

t = time, in days

Integration of equation (6) yields:

$$P = \frac{K_1}{K_2}\,(1-e^{-K_2T}) \qquad (7)$$

where T is the accumulation time in days. Use of equation 7 would limit constituent accumulation to a maximum value of K_1/K_2.

The definition of antecedent dry period as the number of days prior to the storm during which the accumulated rainfall was less than 1.0 inch (DRYDAYS in equation 1) should be further examined. For example, Alley and Ellis (1979) compared simulated total nitrogen loads from two residential areas near Denver, Colorado, with measured loads. They found SWMM to overestimate runoff loads with the extent of overprediction increasing with the value of the model parameter DRYDAYS. DRYDAYS exceeded 30 days in five of the six events simulated, resulting in as much as +1000 percent errors in simulated loads.

Traditionally, equation 7 has been derived with the assumption that urban land surfaces were completely washed by the last cleaning, either mechanical (street sweepers) or by storm runoff. In order to eliminate this assumption, T could be redefined as:

$$T = t + t_o \tag{8}$$

where t is the time since last cleaning, and

$$t_o = - \frac{1}{K_2} \ln \left(1 - \frac{P_E K_2}{K_1}\right) \tag{9}$$

where P_E is the available land-surface load in pounds per unit area of land surface at the end of the last period of street sweeping or storm runoff. Figure 1 illustrates the two options for exponential constituent accumulation, as well as the linear buildup option currently in SWMM. Use of equations 8 and 9 is limited to continuous simulation. However, it would eliminate the need for an arbitrary definition of antecedent conditions.

An offshoot of the pervasiveness of linear accumulation equations in urban runoff models appears to be the reporting of accumulation rates as linear values in pounds per unit area of land surface per day. Even Sartor and Boyd (1972), who first suggested nonlinear accumulation, reported accumulation rates in pounds per curb mile per day. Often the linear rates are determined by dividing the total measured runoff load over a period of time by the contributing area and by the time period in days. Use of such a variable in a runoff-quality model might yield good estimates of annual runoff loads but individual storm-runoff loads could be poorly simulated.

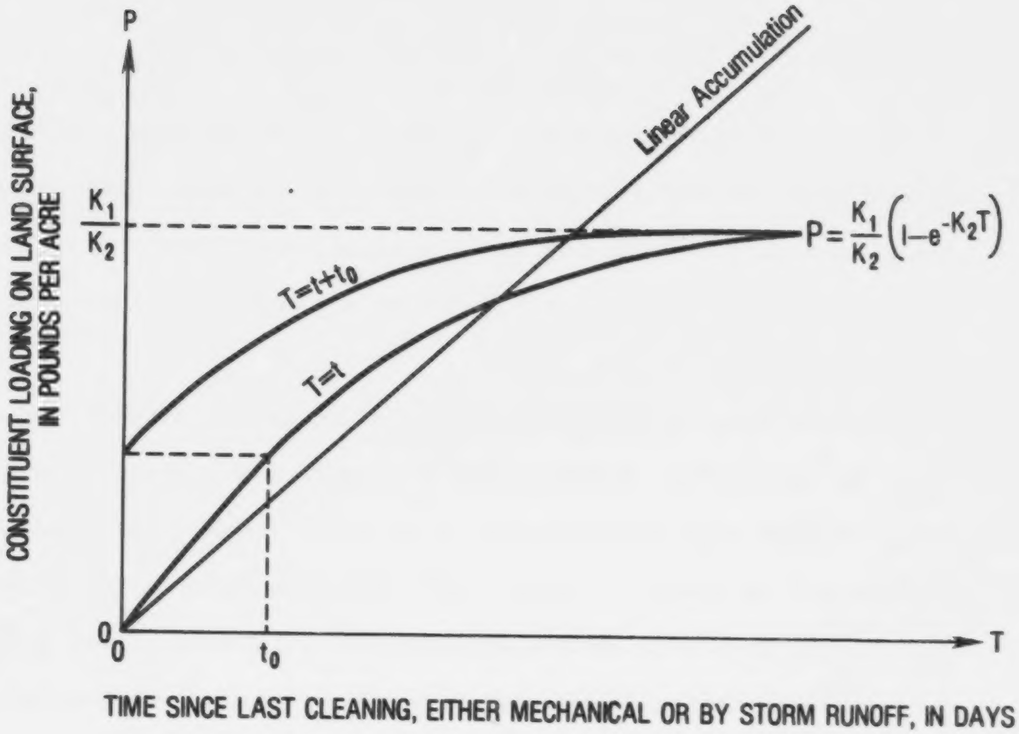

Figure 1.--Constituent accumulation.

In summary, more information is needed on the nonlinear characteristics of constituent accumulation rates on urban land surfaces. Implementation of equations 7, 8, and 9 as an option in urban runoff-quality models such as SWMM might encourage the gathering and reporting of such information.

Constituent Washoff

Several features of the washoff equations warrant further discussion. First, the exponential washoff equation (equation 2) appears to be the subject of some misinterpretation. It is often assumed that equation 2 accounts for the effects of both runoff intensity and duration on water quality. In actuality, because equation 2 is a function of the product of runoff intensity and duration, the amount of constituents washed off during a storm, according to equation 2, is a direct function of the total volume of storm runoff. Equations 3 and 4 are used to account for the effects of runoff intensity.

Generally, the value of α in equation 2 is set at a constant value of 4.6. The rationale for this value is usually stated as--"Assuming that a uniform runoff of 0.5 inches per hour would wash off 90 percent of a constituent in 1 hour." This is a misleading statement in that the actual assumption is that a runoff volume of 0.5 inches will wash off 90 percent of a constituent from urban land surfaces regardless of duration and whether or not the runoff was uniform. Even this statement is only the true assumption if the availability factors ASUS and ASET in equations 3 and 4 are equal to 1.0 for all time steps.

Generally, the value of α is set to 4.6 for all sites and all constituents. However, several studies have reported--the perhaps, not unexpected result--that α varies for different constituents and for different watersheds (Barkdoll, 1975; Smith and Jennings, 1979; and Ellis and Sutherland, 1979). Both Barkdoll (1975) and Sonnen (1979) have found 4.6 to be an overestimate of α in many cases.

The availability factor equations (equations 3 and 4) were derived using constituent accumulation data from Chicago streets and runoff-quality data from Cincinnati. Thus, the coefficients in these equations, if not the form of the equations, could be very site specific. Dustfall data reported for the Chicago and Cincinnati studies show considerable difference (Metcalf and Eddy, Inc., and others, 1971). Thus, the applicability of Chicago constituent accumulation information to a Cincinnati watershed is questionable.

The original formulation of SWMM did not contain the correction factors (F1(I) and F2(I) in equation 5) for insoluble constituents. A value of F1(I) of 0.05 for BOD was somewhat arbitrarily assigned to improve initial testing of SWMM. The rationale for this adjustment was that insoluble BOD was not included in the Chicago data used in developing the model. As other constituents were added to the model, similar corrections were made. When the capability for settleable solids simulation was added to the model, values of the second correction factor F2(I) were assigned. However, inclusion of a settleable solids correction was redundant, since settleable solids are a fraction of suspended solids. Studies by Jewell and Adrian (1978) and by Alley and Ellis (1979) found that the correction for insoluble constituents (equation 5) is the main source of simulated constituent washoff for BOD_5, COD, total nitrogen, and total phosphate. For example, Alley and Ellis (1979) found that application of equation 5 resulted in an average 22-fold increase in simulated total nitrogen washoff over that predicted by the exponential washoff equation (equation 2) alone. Jewell and Adrian (1978) concluded that:

> The original calibration problems appear to have been caused by use of the Chicago data to predict pollutant buildup rather than a need for any insoluble pollutant correction. A later study (Sartor and Boyd, 1972) indicated that pollutant buildup rates for other cities are one to two orders of magnitude greater than those reported in the Chicago study. This alone could explain why initial predicted pollutant washoff was low.

Inclusion of site specific correction factors, such as those embodied in equations 3-5, complicates model calibration and may hinder achievement of satisfactory results. However, equation 2 by itself will often be insufficient for predictive purposes, due to the limitations previously discussed.

An alternate approach could be to eliminate equations 3-5 and include an availability equation for each constituent such as:

$$A(I,L) = c_1(I,L) + c_2(I,L) \times R \qquad (10)$$

where $A(I,L)$ is the availability factor for constituent I on land use L, and $c_1(I,L)$ and $c_2(I,L)$ are coefficients which could be calibrated for each constituent and land use based on local data. Alternately, a term related to shear stress might be included in equation 10 in place of R. Limitations of this approach include difficulties in transferring the coefficients c_1 and c_2 to unsampled watersheds and the fact that constituent availability relationships would be expected to change from event to event and within events due to changes in particle size distribution of constituents.

Another approach might be to replace equations 2-5 with a washoff scheme based on sediment transport theory. An important consideration would be simulation of constituent transport by particle size. Particle-size distributions are not only important for simulating transport of constituents, but also for assessing the effectiveness of management strategies such as street sweeping and detention storage, and for determining impacts on receiving waters.

The U.S. Geological Survey is currently investigating these and other approaches.

Source Identification

An important aspect of runoff-quality modeling is often identification of sources. Two potentially important sources of runoff loads generally have been neglected in urban runoff-quality modeling. These are pervious-area runoff and atmospheric fallout.

Pervious-Area Runoff

The constituent accumulation and washoff equations discussed, thus far, have pertained to impervious-area runoff only. However, the relative contributions of pervious and impervious areas to runoff loads will affect the utility of management strategies such as street sweeping, as well as the outcome of modeling.

A study by Barkdoll, Overton, and Betson (1977) suggests that "semi-pervious and pervious areas are highly significant in their contribution to urban water pollution." Theoretical studies by the originators of SWMM (Metcalf and Eddy, Inc., and others) indicated that "very large rates of runoff would be required to remove dust and dirt from grass plots, and that unless erosion takes place from ungrassed areas, the contribution of pervious surfaces to suspended solids content is minor." However, they noted that runoff from pervious surfaces may contain significant amounts of soluble constituents.

Several indicators of the relative contributions of pervious and impervious areas to runoff loads can be used. These include empirical knowledge of sources, results of regression analyses, distributions of constituent concentrations and loads over storm hydrographs, results of street-surface sampling, and the effectiveness of management strategies such as street sweeping.

Empirical knowledge of sources can provide an indication of the relative contributions of pervious and impervious areas to runoff loads for certain constituents. For example, one might expect pesticides to originate primarily from pervious areas and lead which is associated with automobile exhaust to originate primarily from impervious areas. Other constituents such as nitrogen species which are associated with organic pollution, fertilizers, and automobile sources may originate primarily from either pervious or impervious areas.

Regression analyses of storm-runoff loads with parameters such as average daily traffic or pavement condition may provide an indication of sources by identifying significant variables.

The distribution of constituent concentrations and loads over storm hydrographs may provide some indication of the relative contributions of pervious and impervious areas to runoff loads. For example, the decay coefficient (α) in equation 2 would be expected to be smaller for constituents with a larger pervious-area contribution to runoff loads. In addition, the relative proportions of pervious- and impervious-area contributions to runoff, as determined from a rainfall-runoff model, might be compared to trends in constituent concentrations and loads.

Sampling of street-surface solids in conjunction with runoff-quality studies may provide a useful indication of the potential impervious-area contributions to storm-runoff loads. A limitation of street-surface sampling is that it provides an estimate of the total accumulation of constituents on the street surface, but not of the street-surface load "available" for washoff by storm runoff. However, analysis of constituent partitioning amongst street-surface solids by particle size can provide an indication of constituent availability for washoff.

Finally, the effectiveness of management strategies such as street sweeping may provide an indication of the relative importance of pervious and impervious areas as sources of storm-runoff loads. For example, effectiveness of street sweeping as measured by removal of constituents from streets could be compared to its effectiveness in reducing constituent loads in storm runoff.

Due to rapid response times of most urban watersheds, of primary interest would be surface runoff and quick-return flow (i.e., interflow with shallow penetration of the soil). Chemical constituents added to solution would normally be those characteristics of surficial soils.

Unfortunately, techniques for predicting the interactions between these soils and the runoff are not far advanced. Given the state-of-the-art, simple empirical equations for pervious-area runoff might be included in the model. For example, Leonard and others (1979) report simple power functions decribing soil-based herbicide transfer to runoff. They also note that average storm herbicide concentrations in runoff were correlated with herbicide concentrations at the 0- to 1-centimeter depth increment of the watershed soils at the time of runoff.

Atmospheric Fallout

Most atmospheric fallout samples have been collected with devices that remain open continuously. Generally, analyses of such samples are reported as bulk precipitation. Bulk-precipitation data provide measures of the dissolved material in wet precipitation (wetfall) plus water-soluble materials that have been leached from the dry fallout (dryfall) as the sample awaited processing. From the modeling standpoint, these data are more useful if wetfall and dryfall are separated. The data are also more useful if the water-insoluble components are included in the wetfall and dryfall analyses. Various studies have suggested that both wetfall and dryfall may be important sources for many constituents in runoff from urban areas (Betson, 1978; and Barkdoll, Overton, and Betson, 1977).

Urban runoff can be viewed as chemically modified rainwater. Accounting for the original chemical composition of this rainwater may be important in terms of both model reliability and evaluation of management practices. For example, wetfall contributions to runoff loads would be unaffected by street sweeping practices. The simplest means of accounting for wetfall contributions to runoff loads in a model would be to add a concentration representing wetfall to the concentrations of a constituent predicted by the washoff equations. This adjustment could be constant or could vary according to season. Although it is well known that the chemistry of wetfall can change within a particular storm, few data exist to quantify this effect. Thus, a simple adjustment might be all that is currently justified. Certainly, more wetfall data in conjunction with runoff-quality data are needed.

Dryfall data are generally collected without consideration of resuspension. Thus, dryfall data could provide an indicator of the value of K_1 in equation 7. Ideally, dryfall data may provide a basis for transferring constituent accumulation information from one urban watershed to another. As for wetfall, seasonal values for dryfall indicators might be used.

SUMMARY AND CONCLUSIONS

An analysis of the SWMM surface-runoff-quality algorithms has been presented. Limitations of the constituent accumulation equations include the assumption of linear buildup of constituents on urban surfaces with no upper limit on the amount accumulated, the arbitrary antecedent-conditions criterion (DRYDAYS), and the assumption of completely clean urban surfaces at the start of the DRYDAYS period. An alternative approach which eliminates these assumptions has been presented.

Several features of the washoff equations were also discussed. These included the misinterpretations often resulting from use of the exponential washoff equation (equation 2) and the limitations of the availability factor equations (equations 3 and 4) and the insoluble correction factor equation (equation 5). A different approach might include deletion of equation 5 and substitution of availability factor equations for each constituent in place of equations 3 and 4. Either a runoff rate or shear stress term might be included in these availability factor equations. Alternately, a sediment transport approach might be used.

Finally, the need to address the contributions of pervious-area runoff and atmospheric fallout to runoff loads was discussed.

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THE USE OF SWMM TO ECONOMICALLY MODEL SURCHARGED COMBINED SEWER SYSTEMS

by

WILLIAM M. PARKER III[1]

The Village of Skokie, Illinois (pop. 70,000) is a near-northern suburb of Chicago approximately 10 square miles in area. The Village owns and operates a combined sewer collection system tributary to the Metropolitan Sanitary District of Greater Chicago (MSDGC) interceptor and treatment system. Intercepted flows are treated at the MSDGC's Northside Sewage Treatment Works, located in Skokie. Flows in excess of the interceptor system capacity are discharged to the nearby North Shore Channel. An area map relating the above-described features is shown in Figure 1.

The combined sewer and interceptor systems each have limits which are exceeded during certain precipitation events. For many sections of Skokie, the collection system limit is exceeded several times each year. The result is that combined sewage backs up into basements and onto streets on a Village-wide basis and is not restricted merely to downstream portions of the collection system near the North Shore Channel. The MSDGC interceptor system capacity is currently exceeded over 90 times each year and the resulting water pollution problems have a major regional impact, as there are 53 other suburban communities connected to the MSDGC system.

The MSDGC has adopted and is now implementing the Tunnel and Reservoir Plan (TARP), which will essentially increase systemwide interceptor capacity and limit overflows

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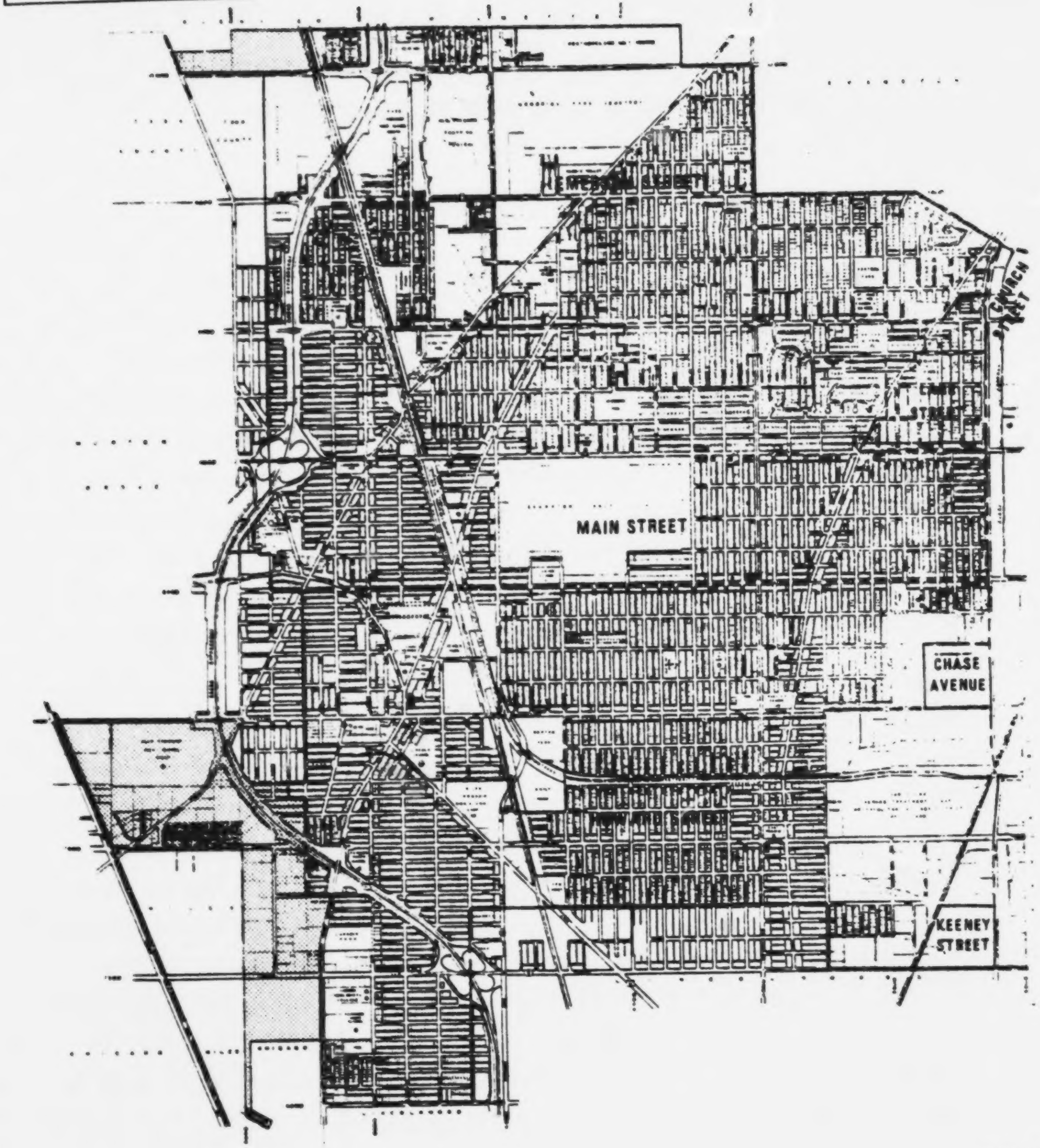

EMERSON STREET
CHURCH STREET
MAIN STREET
CHASE AVENUE
KEENEY STREET

LEGEND

Subdistrict Boundary — - — -

Area Serviced By
Other Sewer District ▨

NORTH

0 1 2
Scale In Thousands Of Feet

Figure 1
SUBDISTRICT DELINEATION
COMBINED SEWER CONVEYANCE PROBLEMS
SKOKIE • ILLINOIS

to local waterways. The system components of TARP are designed to accomodate improvements in the local collection systems, such as Skokie's, by providing a hydraulic outlet for the 5-year rainfall event. Thus, while the MSDGC is proceeding to improve the outlet conditions for the local collection systems, the initiative to correct remaining local system inadequacies remains with each of the tributary communities.

A study [1/] has recently been completed for the Village which addresses local system inadequacies and has recommended alternative corrective measures to upgrade the Skokie system in order to obtain the full benefit of TARP. The RUNOFF and EXTENDED TRANSPORT blocks of the U.S. EPA's Stormwater Management Model (SWMM) were selected to conduct extensive computer simulation of the existing collection system. The EXTENDED TRANSPORT block of SWMM was selected to be used rather than the conventional TRANSPORT block so that simulated water surface vs. time data could be calibrated and verified against similar data collected in the field.

Model Set-up and Development

The Skokie collection system consists of three major sub-districts served by large diameter trunk sewers running in an east-west alignment toward the North Shore Channel and the MSDGC interceptor. The Mainstream Tunnel component of TARP runs directly below this interceptor and will accept flows from Skokie in excess of the interceptor capacity at the points which the three major trunk sewers join the interceptor. Three catchments were selected to be simulated with SWMM, each corresponding to a major trunk sewer sub-district, namely the Emerson, Main, and Howard Street catchments.

The Howard Street catchment was selected to be monitored with four, continuous water surface vs. time recorders placed in key manholes. The Howard Street catchment was chosen in this regard principally because it contained a 105 acre area which had previously been studied by the Village Engineering Department.[2/], [3/] This area, known as Fairview South, had been the subject of a Village initiated rooftop downspout disconnection program and offered a comprehensively developed data base. Fairview South, therefore, was selected as the area to develop the most economically suitable fashion to apply SWMM on a Village-wide basis and obtain reasonable simulation results.

Because 1) system surcharging is a Village-wide problem in Skokie, and 2) it was not practical to simulate the entire collection system due to model limitations and economics, lateral and branch sewers were necessarily simulated as gutter/pipes in the RUNOFF block for the overall sub-district simulations. Accurate characterization of gutter/pipes to best represent lateral and branch sewer performance under surcharged conditions was therefore necessary to develop realistic alternative correction actions. This need resulted in the study of a two block length of Lunt Avenue within Fairview South at two levels of detail to accurately characterize gutter/pipe capacities for larger scale, Village-wide assessment.

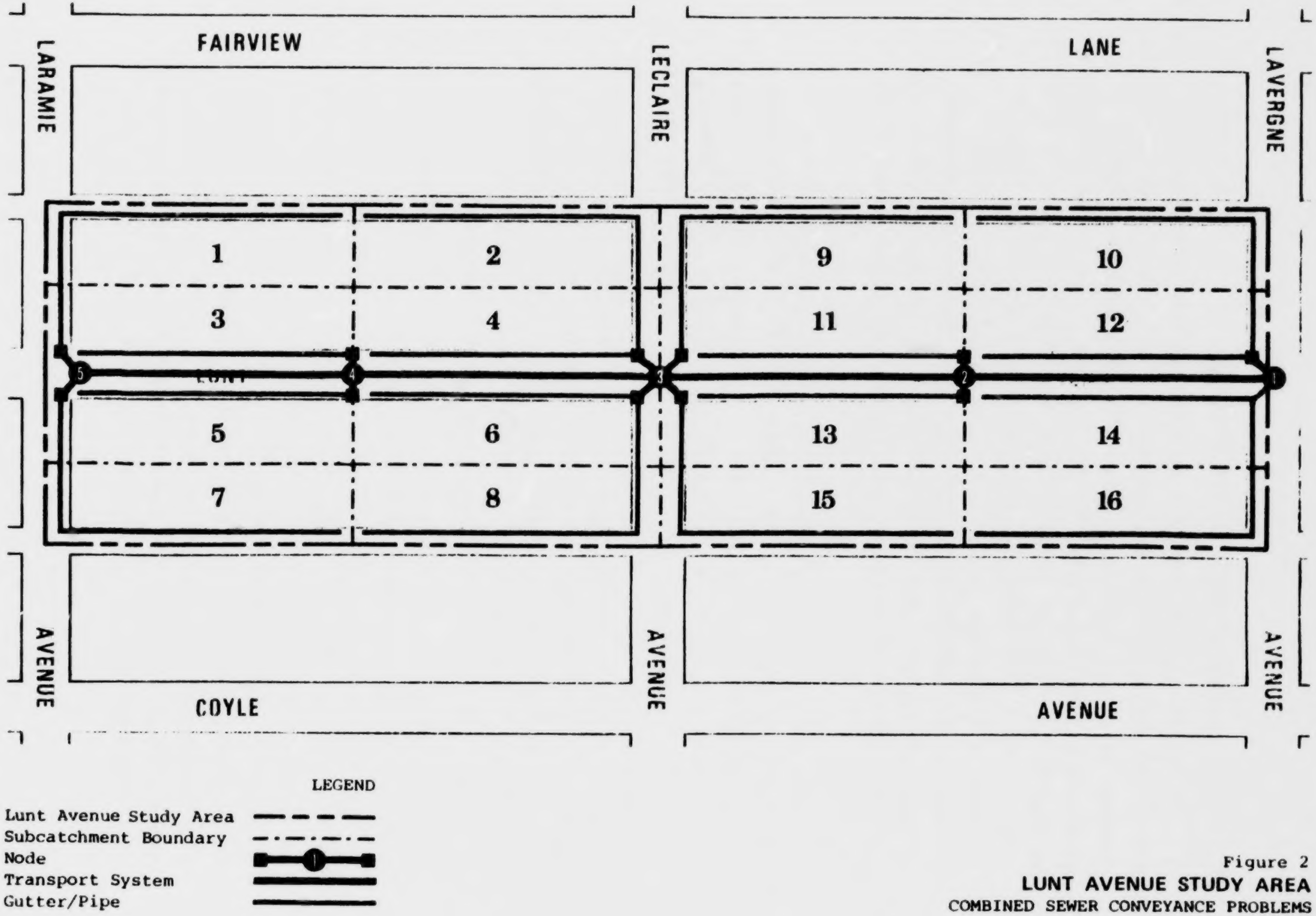
An additional concern was the loss of model resolution and possible accuracy when model sub-catchments were increased in size from Lunt Avenue (10 acres) to Fairview South (105 acres). This concern prompted an additional sensitivity analysis of key RUNOFF block parameters on two levels of detail for the entire Fairview South area. A description of each effort follows.

Characterization of Gutter/Pipe Capacity

The Lunt Avenue area was modeled at two levels of detail to determine the sensitivity of the results to changes in gutter/pipe capacity. The gutter/pipe capacity, which is implicitly stated in the RUNOFF block of SWMM, is specified in terms of slope, diameter, and friction coefficient. The gutter/pipe capacity is important because it affects the shape and peak of the hydrograph that is routed in the EXTENDED TRANSPORT block. Accurate simulation of the hydrograph that is to be routed was fundamental to the sizing of relief conveyance facilities for the Village trunk sewers.

The Lunt Avenue area consists of two typical blocks in the Fairview South area. The land use in this 10 acre area is all single-family residential. As shown on Figure 2, the area drains to catch basins which direct runoff to a street lateral. This lateral is approximately 1200 feet long with an average slope of 0.0026 and an outlet diameter of 18 inches. Based on a Manning's "n" value of 0.015 and a slope of 0.0055 in the last downstream pipe section, the lateral outlet capacity under full flow, gravity conditions is 6.7 cfs. This outlet capacity is also based on the assumption that the trunk sewer is not surcharged.

It appears that this street lateral has inadequate capacity to convey the runoff from an event with a 5-year recurrence interval. Based on the Chicago Hydrograph Method by Kieffer and Tholin[4/], the required outlet capacity would be about 15 to 18 cfs for the 5-year event. The apparent inadequacy of the lateral sewer capacity indicates that surcharging during the 5-year event will occur even with no surcharging in the downstream branch or trunk sewers. If surcharging occurs in the lateral sewer, then the hydrograph from the Lunt Avenue lateral will have a peak flow for the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LANE
LARAMIE
LECLAIRE
LAVERGNE
LUNT
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2
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12
13
14
15
16
AVENUE
AVENUE
AVENUE
COYLE
AVENUE
LEGEND
Lunt Avenue Study Area
Subcatchment Boundary
Node
Transport System
Gutter/Pipe

Figure 2
LUNT AVENUE STUDY AREA
COMBINED SEWER CONVEYANCE PROBLEMS
SKOKIE • ILLINOIS

5-year event greater 6.7 cfs. To properly size downstream relief facilities it appears that the lateral (gutter/pipe) capacity should be specified as greater than 6.7 cfs.

The most detailed characterization of the Lunt Avenue area was used to evaluate the problem of specifying the gutter/pipe capacity. For this most detailed characterization, the Lunt Avenue Area was delineated into 16 sub-catchments and the RUNOFF and EXTENDED TRANSPORT blocks set up to model the two block long sewer system.

The detailed characterization provided an opportunity to simulate the routing of flow through the street lateral. Results of this analysis were then compared to the results of the coarser Lunt Avenue characterization, for which the area was modeled with the RUNOFF block as a single sub-catchment and the lateral sewer was represented as a gutter/pipe (with no routing). Because the cost and time involved in characterizing the entire study area at the same level as the more detailed Lunt Avenue characterization was prohibitive, the coarser characterization required adjustments so that its results would closely approximate the more detailed characterization results. By this comparison, a means of calculating gutter/pipe capcity for larger areas was developed.

The more detailed level of modeling tended to exaggerate the effect of downspout disconnection. To ensure that the results of this analysis were reasonably representative, two conditions were identified for the RUNOFF block. Condition 1 was set up assuming that all roof area was pervious (downspouts disconnected), resulting in a total impervious area of 17 percent. Condition 2 represented the situation wherein the roof areas were impervious and connected directly to the lateral sewer, resulting in a total impervious area of 36 percent.

Four simulation runs were made for the more detailed characterization of the Lunt Avenue area. Conditions 1 and 2 were each run for the 2-year and 5-year (both 2-hour) rainfall events, and the four outlet hydrographs are shown in Figure 3. The 5-year event, as simulated, caused surcharging and indicated that for both Conditions 1 and 2 the peak flow to an unsurcharged downstream sewer would exceed 6.7 cfs (full flow capacity of the 18-inch lateral). Also shown on Figure 3 are two hydrographs from the coarser characterization. These hydrographs are for the 2-year recurrence interval event and indicate (from the flattened peak) that a specified gutter/pipe capacity of 6.7 cfs constrains the flow. By setting the outlet capacity on the basis of having surcharged conditions (in effect, increasing the gutter/pipe slope), it appears that a more representative hydrograph is determined.

Neither Condition 1 nor Condition 2 quite accurately represents roof areas being disconnected or connected, respectively. The RUNOFF block simulates various abstractions (rainfall losses) from pervious areas. Considering roof area to be pervious in Condition 1 allows the model to abstract too much rainfall from an artificially designated pervious area. Thus, the resultant modeled outlet hydrograph peak flow and total runoff volume estimate are lower than would be expected to actually occur. In Condition 2, the model routes roof area runoff by gutters to a catch basin rather than entering the runoff directly into the lateral sewer. Consequently, the resultant modeled outlet hydrograph peak flow may be inaccurate. The actual hydrograph for Condition 1 is estimated to fall between the two modeled hydrographs.

Inspection of the simulated hydraulic gradeline along Lunt Avenue indicated that even with free outfall conditions to the branch sewer along Lavergne Avenue, surcharging

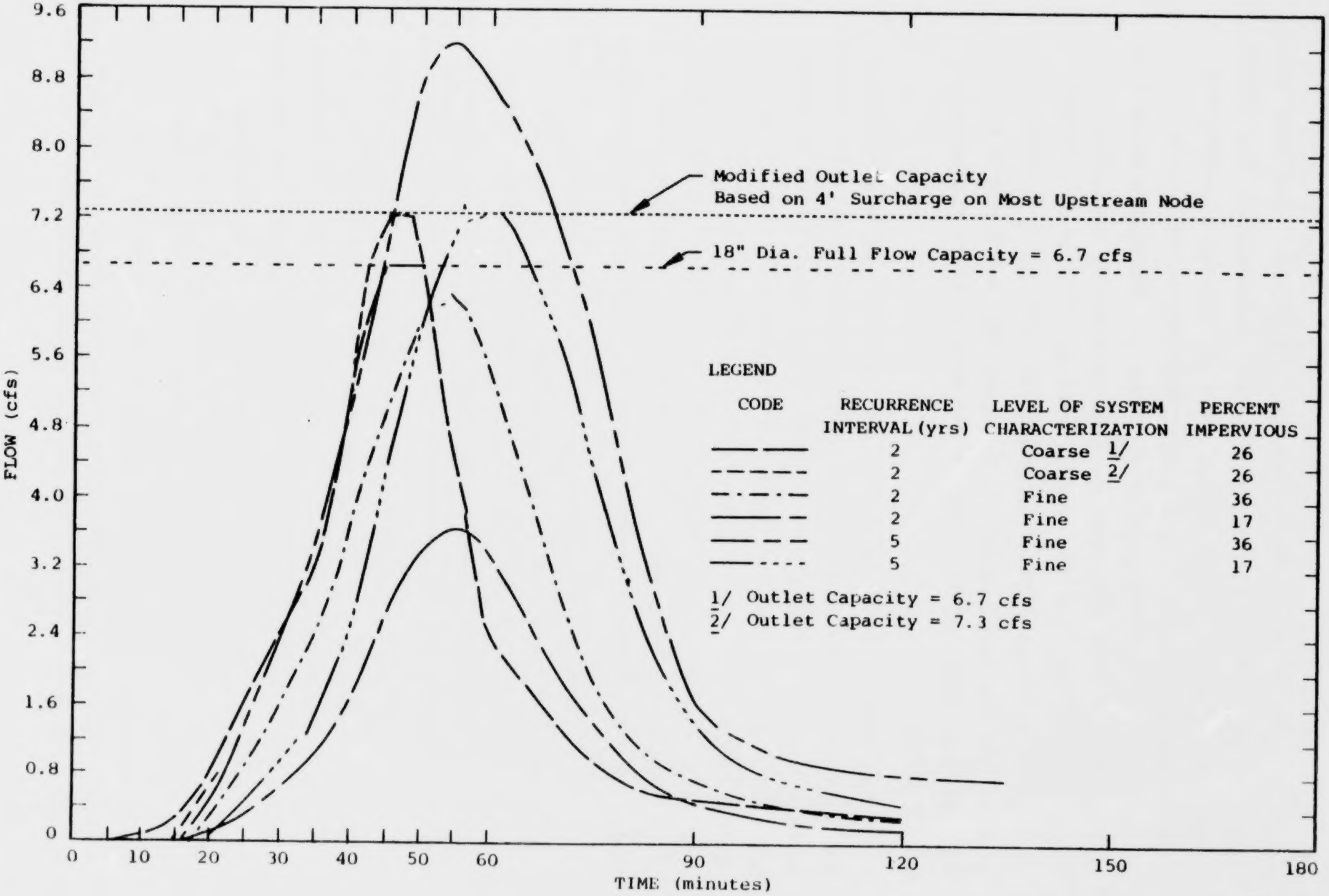


Figure 3

OUTLET HYDROGRAPHS

LUNT AVENUE AREA CHARACTERIZATION

COMBINED SEWER CONVEYANCE PROBLEMS

SKOKIE • ILLINOIS

would occur and could cause basement backups. For the 5-year event, Condition 2 (rooftops directly connected), the hydraulic gradeline rose to within three feet of the ground surface midway along the two block length and was within only several inches of the ground surface at the upstream end of the lateral. The duration of the surcharged condition is about 30 minutes for the 5-year event. The 2-year event, Condition 2, also caused surcharging, but to a smaller degree, as the most upstream manhole was surcharged to within four feet of the ground surface for approximately 10-15 minutes.

Two conclusions were reached in studying the analysis of the Lunt Avenue area. One conclusion is that the lateral sewers are not adequate to convey the 5-year event without surcharging. Thus, improvements to increase the trunk sewer and branch sewer conveyance capacity would not entirely eliminate surcharging in the laterals. The second conclusion was that the gutter/pipe capacity should be specified to reflect surcharged conditions in the laterals.

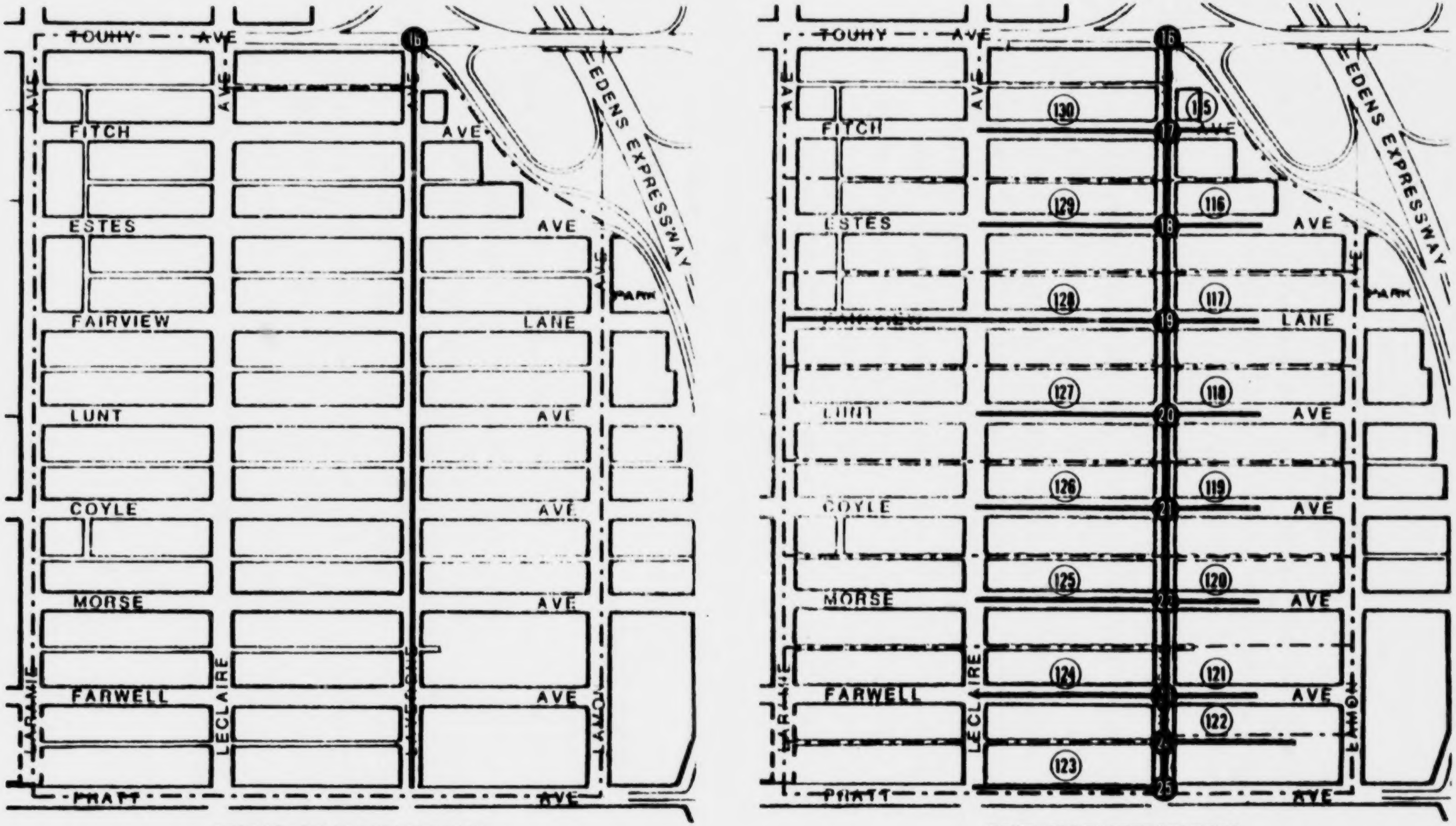
The extent of simulated lateral surcharging is insignificant relative to actual conditions if the trunk and branch sewer do not surcharge. When the trunk sewers surcharge, the duration, magnitude, and extent of lateral surcharging is greatly increased.

Evaluation of the Lunt Avenue Area indicated that the gutter/pipe capacity should be calculated for the Village-wide analysis as 0.9 cfs/acre tributary to EXTENDED TRANSPORT block nodes. The Skokie sewer system was checked to determine whether or not a parametric analysis was applicable for specifying outlet capacity of each gutter/pipe. Most laterals appeared to be sized similarly to the Lunt Avenue lateral and were expected to have similar capacity limitations. Thus, for 10 acre (two block) sub-catchments, the outlet sewer capacity was calculated to provide 0.9 cfs/acre for each gutter/pipe. Based on overall similarity of the various lateral systems, this quantity was subsequently used for gutter/pipes in all sub-catchments rather than individual analysis of the existing outlet pipe for the sub-catchment.

Sensitivity to Other Runoff Parameter

Necessary data for the RUNOFF and EXTENDED TRANSPORT blocks of SWMM were developed on two levels for the Fairview South Area. Initially, the area was broken down into 16 sub-catchments of approximately 10 acre size (fine). Each sub-catchment represented a two block area approximately similar to the Lunt Avenue Area. Laterals through these sub-catchments were represented by the gutter/pipe routine of the RUNOFF block, with the branch sewer (running north and south down Lavergne Avenue) modeled in the EXTENDED TRANSPORT block. The Fairview South Area was also delineated as a single 105 acre sub-catchment (coarse) for use as part of the entire Howard Street catchment modeling effort. Figure 4 illustrates the two characterizations of the Fairview South Area.

The Fairview South Area was as a calibrating link between large scale, 100 acre sub-catchment modeling and the smaller, 10 acre size approach, which was represented by the Lunt Avenue area analysis. Investigations were made to determine the sensivity of various model output parameters to several varied input parameters in the RUNOFF block necessarily associated with increased coarseness of drainage area representation. Various outlet hydrographs resulting from the application of the 10-year, 2-hour storm on the Fairview South Area are shown in Figure 5. The conditions producing each hydrograph are also shown in Figure 5.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ESTES AVE
FAIRVIEW LANE
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COARSE CHARACTERIZATION

FINE CHARACTERIZATION

LEGEND

Catchment Boundary	— · — ·
Node	●
Transport System	▬
Gutter/Pipe With Identifying Number	(111)

Figure 4

CHARACTERIZATION OF FAIRVIEW SOUTH AREA

COMBINED SEWER CONVEYANCE PROBLEMS

SKOKIE • ILLINOIS

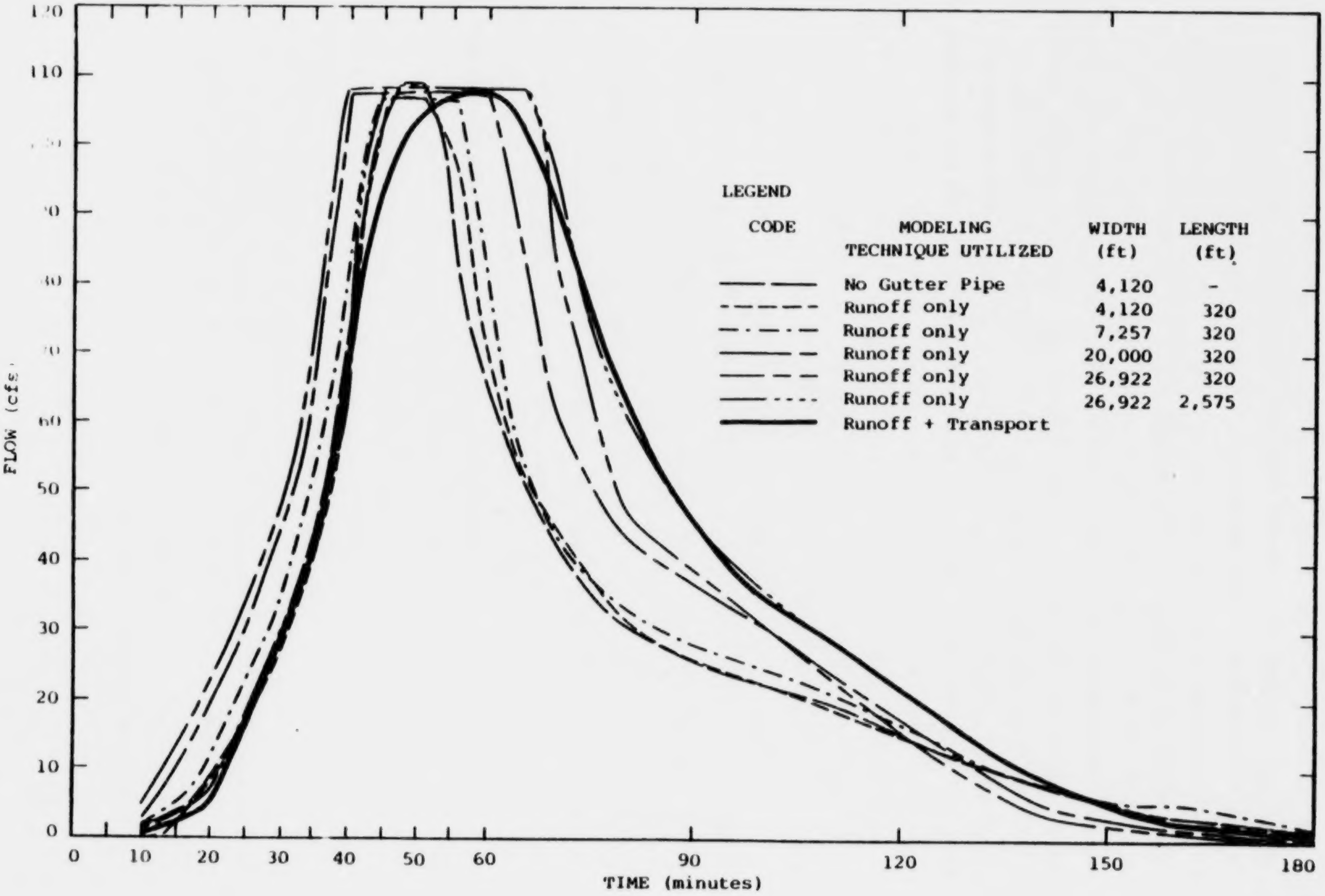

Figure 5

OUTLET HYDROGRAPHS • 10 YEAR • 2 HOUR STORM
FAIRVIEW SOUTH CHARACTERIZATION
COMBINED SEWER CONVEYANCE PROBLEMS
SKOKIE • ILLINOIS

Through this sensitivity analysis, it was found that the total volume of runoff was most sensitive to the width of sub-catchment. The width of sub-catchment is an arbritary measure of the total width of overland flow and is commonly approximated by double the length of the longest (main) drainage conduit through the sub-catchment, since two plane catchments contribute flow along this length. It was found that as the size of the sub-catchment increased, the width of sub-catchment lost its significance as previously described. Additionally, as the size of the sub-catchment was increased, more rainfall abstraction was simulated than had been anticipated. This is thought to be due to a relatively large increase in the length of overland flow for increased sub-catchment sizes without a corresponding increase in the width of sub-catchment.

Model output, specifically the shape of the outlet hydrograph, was found to be fairly sensitive to land slope, as evidenced when default values for the parameter were replaced with measured quantities. Model output was not found to be excessively sensitive to the other input parameters tested, namely imperviousness, overland flow "n" values, or infiltration rates. As such, input quantities were determined from existing data where available. Where existing data were not available, default values recommended by the SWMM User's Manual [5/] were used.

It was determined from efforts to match hydrographs for the coarse and fine representations of the Fairview South Area that the width of sub-catchment could reasonably be expressed as a function of sub-catchment size. The following relationship was therefore used to determine sub-catchment widths for other sub-catchments in the study.

(1) Width of sub-catchment (feet) = 250 ft/acre x sub-catchment acreage

Calibration and Verification

The general purpose of the field studies conducted for the Skokie study was to gather data concerning the existing sewer system and factors that affect the quality, amount, and rate of runoff to the sewer system. More specifically, field data were obtained to permit comparison of actual and predicted hydraulic responses of the sewer system to several precipitation events. The comparison of field data and model output showed that the model produces representative results within reasonable accuracy and verified the conclusions derived during the analysis of model results.

Monitoring and Data Collection Program

Field studies were limited to the Howard Street catchment. The Howard Street catchment was chosen as representative of a typical drainage area within the Village. Additionally, included within this sub-district is the Fairview South Area, unique in the respect that approximately 85 percent of the roof downspouts have been disconnected from the sewer system.

Rainfall data for storms that occurred during the flow monitoring program were necessary for calibration of SWMM and verification of the program once calibration was completed. Rainfall data were collected for the storms of June 5, 8, 11, 17, 24, and 30, 1977 from the MSDGC. The MSDGC maintains a rainfall gauge at their Northside Sewage Treatment Works, located in southeastern Skokie at the intersection of Howard Street and McCormick Boulevard.

Field data were also collected with regard to Manning's "n" values at various points within the Howard Street sewer system. These data were obtained to provide representative input data to the EXTENDED TRANSPORT block. Roughness coefficients were calculated based on several different field velocity and depth of flow measurements as well as other hydraulic properties of the particular sewer section.

Continuous measurement of water surface elevations at selected points within the Howard Street sewer system was also included in the field studies program. These measurements were conducted for about 45 days to obtain data during the six before-mentioned precipitation events. Information collected from four of these events was used for calibration of SWMM. Two of these events were used for verification. No quality measurements were taken, as only the capacity of the system was investigated in the field monitoring program.

Four recorders were used for continuous monitoring of water surface elevations. One unit was located in the Fairview South Area to check the response in the sewer system to the control measures previously mentioned. This unit was located in the manhole at the corner of Fitch Street and Lavergne Avenue. Another recorder was located at the corner of Howard Street and Lavergne Avenue to monitor the area downstream of Fairview South. The third and fourth recorders were located in manholes along Howard Street in the main trunk sewer itself. The locations along Howard Street provided data representative of the system response to runoff from both the total and intermediate areas.

Presentation of Results

The following section presents information on rainfall and runoff for events observed in the field during the course of the study. Six independent rainfall/runoff events were observed during the field study program. Pertinent data concerning these six events are summarized in Table 1.

Table 1
DATA FOR FIELD MONITORED STORMS

Date	Total Rainfall	Maximum Intensity[1]	Duration of Rainfall	Recorders Operational[2]
June 05, 1977	0.95 in.	0.44 in./hr.	4 1/2hrs.	1,3,4
June 08, 1977	0.32 in.	0.60 in./hr.	1/2hr.	1,3,4
June 11, 1977	1.90 in.	0.44 in./hr.	3/4hrs.	1,3,4
June 17, 1977	0.77 in.	0.42 in./hr.	5 3/4hrs.	1,4
June 24, 1977	0.20 in.	0.12 in./hr.	1hr.	1,2,3,4
June 30, 1977	1.49 in.	0.70 in./hr.	7hrs.	1,2,3

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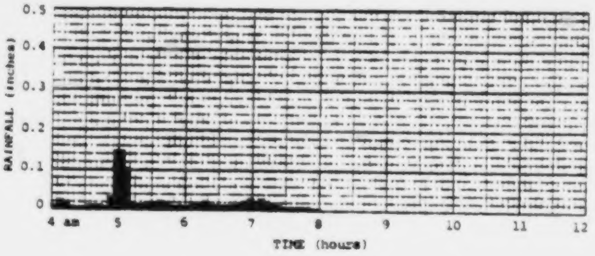
[1] Intensities for a 15 minute duration.

[2] Key to Recorders:

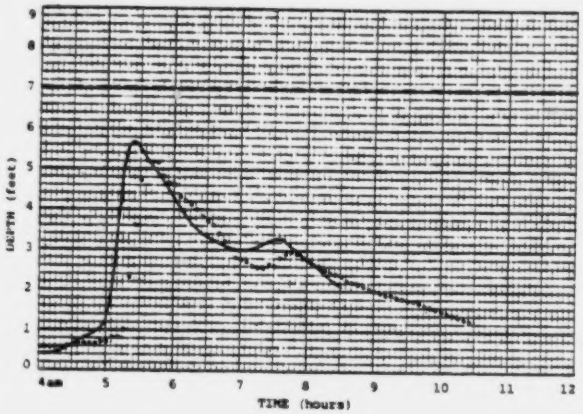
1	Howard and CNW RR	(node 2)
2	Howard and Kenneth	(node 8)
3	Howard and Lavergne	(node 12)
4	Fitch and Lavergne	(node 17)

The events of June 05, 08, 11, and 17, 1977 were used for calibration purposes. The events of June 24 and 30, 1977 were used to verify the model. Representative calibration and verification plots are presented in Figures 6 and 7, respectively.

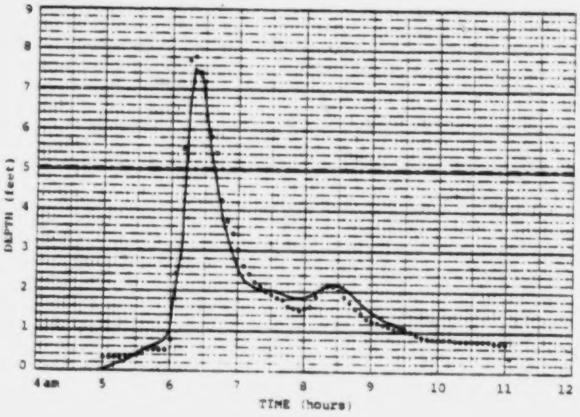
The calibration process pointed out the sensitivity of the gutter/pipe capacity to the results of the simulation. It appeared that surcharging in the trunk sewers caused the sub-catchment outlet hydrograph to be affected. The simulated outlet hydrograph is also affected by the specified characteristics of a gutter/pipe. During surcharging of the trunk sewer the actual hydraulic gradelines on the lateral and branch sewers will be less than the pipe slope at certain times. Thus, the actual outlet peak flows might



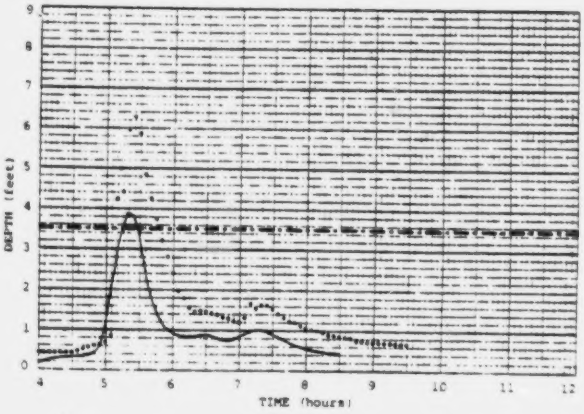
RAINFALL HISTORY



NODE 2



NODE 12



NODE 17

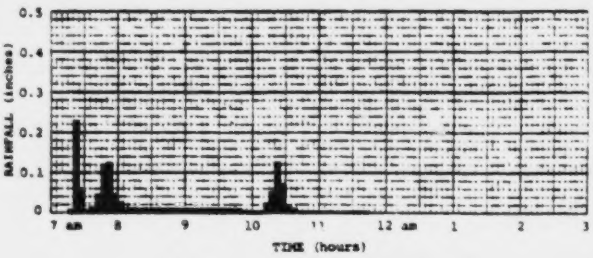
LEGEND

Field Observations ···········

Simulation Results ————

Sewer Crown Elevations -—-—-

Figure 6
JUNE 5 • 1977
WATER SURFACE ELEVATIONS
COMBINED SEWER CONVEYANCE PROBLEMS
SKOKIE • ILLINOIS



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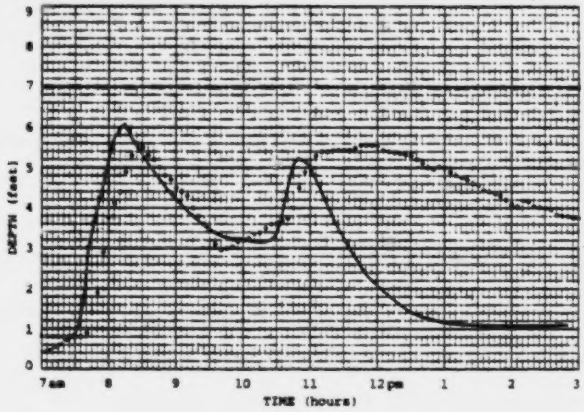
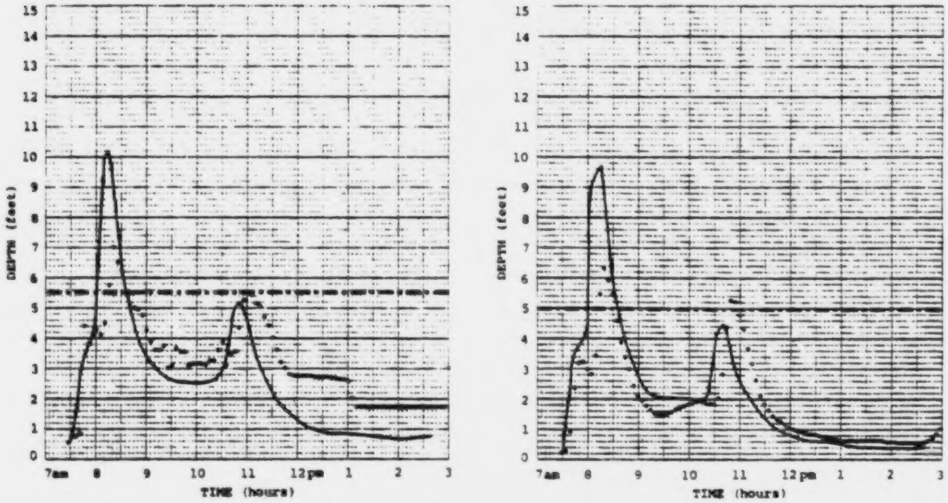


NODE 2



NODE 8

NODE 12

LEGEND

Field Observations ·····················

Simulation Results ————

Sewer Crown Elevations -·—·—·-

Figure 7

JUNE 30 • 1977

WATER SURFACE ELEVATIONS

COMBINED SEWER CONVEYANCE PROBLEMS

SKOKIE • ILLINOIS

be less than those simulated by the RUNOFF block of SWMM. Because of program limitations, it is not possible to adequately compensate for the varying gutter/pipe outlet conditions. Thus, for each event a specific gutter/pipe slope was selected with all other parameters held constant to obtain a reasonable match between observed and simulated results.

For this study, it was most important that the simulated results match the observed depth measurements at the most downstream recorder. The response simulated for this location is less subject to errors caused by limitations the model has in simulating runoff from all the sub-catchments. Over the full length of the modeled system, these limitations tend to balance out resulting in a reasonable representation of routed flow. In contrast, the comparison of simulated results and observed measurements at the recorder located in Fairview South is affected to the greatest degree by the modeling limitations.

Summary

Flows in the Village of Skokie combined sewer collection system were modeled with the RUNOFF and EXTENDED TRANSPORT blocks of SWMM for six observed precipitation events. Model set-up and development indicated that 1) gutter/pipe characterization, and 2) width of sub-catchment were key parameters in modeling the system because of Village-wide surcharging experienced during frequent rainfall events. Accurate representation of system flows was possible at a computer run time of about 0.06 CPU/acre modeled for a 2 hour storm.

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IMPROVEMENTS IN EXTRAN

by

Larry A. Roesner, Atef M. Kassem, and Paul E. Wisner

WRE-TRANSPORT MODEL

- The original WRE-TRANSPORT model was developed in 1974 by CDM/Water Resources Engineers. The formulation of the model is contained in the EXTRAN Documentation.

- A conceptual presentation of the model is shown in Figure 1. The model is based on a link-node description of the sewer system. Links represent pipes and flow diversions. The nodes correspond to manholes or pipe junctions in the physical system.

- The model is based on a complete solution of the gradually varied, unsteady free surface flow equations (Saint-Venant equations). The numerical integration is accomplished by a modified Euler method - for details on the numerical solution refer to EXTRAN Documentation.

- The model has the following capabilities:
  - computation of surcharges and backwater effects;
  - computation of flow diversions by orifices, weirs, pumps, etc.;
  - it accepts inflow-hydrographs by direct input from cards or hydrographs computed by the SWMM-RUNOFF routine.

- The model has been extensively applied in Canada and the U.S. However, many users have experienced stability problems under certain conditions, particularly with weirs, orifices, large drops, or superpipes (usually used for underground storage) with restricted release.

- Subsequent to the release of the original WRE-TRANSPORT model, many modifications were incorporated by various users, many times without consulting the model developers. Consequently, a large number of modified versions are currently used by different consultants and organizations. Not surprisingly, comparative analysis carried out in

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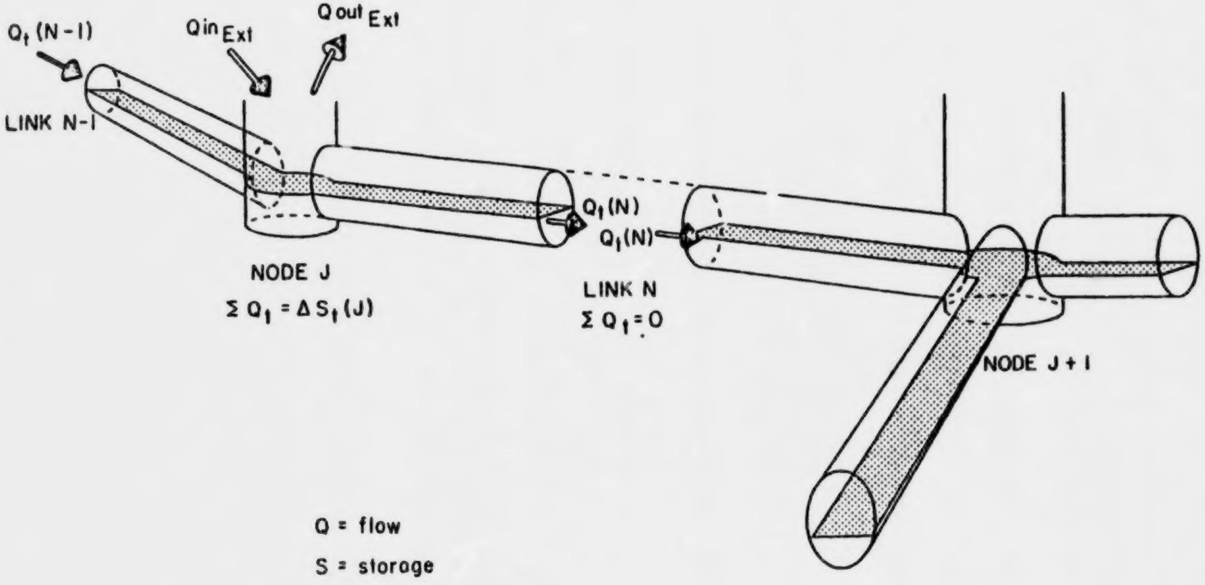

$Q_t(N-1)$
Qin_{Ext}
$Qout_{Ext}$
LINK N-1
NODE J
$\Sigma Q_t = \Delta S_t(J)$
$Q_t(N)$
$Q_t(N)$
LINK N
$\Sigma Q_t = 0$
NODE J + 1
Q = flow
S = storage

FIGURE 1

CONCEPTUAL REPRESENTATION OF THE TRANSPORT MODEL the frame of IMPSWM project indicated differences between results obtained from various versions when tested under the same conditions.

RECENT IMPROVEMENTS OF WRE-TRANSPORT MODEL

- The latest version of the WRE-TRANSPORT model represents a major improvement of surcharge computation. It also has a better print-out. Preliminary tests showed that the program is stable in most cases. The following outlines the features of the new program:

  (a) improved surcharge computation

  (b) weir transfers are node to node, which improved stability;

  (c) print-out summary for junctions, which shows:

    - maximum computed depth and time of occurrence;
    - maximum computed surcharge;
    - minimum depth below ground elevation;
    - duration of surcharge.

  (d) print-out summary for conduits which shows:

    - design flows;
    - maximum computed flows and velocities and their time of occurrence;
    - ratio of maximum to design flows;
    - maximum depth above inverts at conduit ends.

- However, the new version did not completely solve the instability problems with orifices and weirs. Improvement of stability under these conditions is currently undertaken jointly by WRE and IMPSWM.

PRESENT RESEARCH ON "IMPROVEMENT" OF WRE-TRANSPORT

- An agreement is reached between the CDM/Water Resources Engineers and IMPSWM to work jointly on the improvements of the WRE-TRANSPORT in order to eliminate the remaining problems with the model.

  (a) Improvements of stability with orifices:

    - two directions are followed for improving the orifice simulation: (i) an approach similar to the one presented in the current release, and (ii) by converting the orifice into equivalent pipe, which is done internally in the model;

- two options of the orifice are incorporated in the model: (i) bottom orifice, and (ii) side orifice;

- preliminary tests with the orifice simulated as an equivalent pipe indicate good results.

(b) Improvements of stability with weirs:

Although the weirs are seldom unstable, modelling of weirs by means of equivalent conduits is being investigated.

(c) Computation of surface area in order to eliminate any artificial storage. The proposed scheme is explained in Figure 2. With this scheme, one should avoid large drops (high pipe). However, work continued to account for this condition;

(d) Improvements of modelling on-line underground storage (example, superpipe).

(e) Some improvements of printout.

SURCHARGE COMPUTATION BY THE WRE-TRANSPORT MODEL

- All the previous versions of the WRE-TRANSPORT simulate surcharge based on an analogy with a surge tank (storage node).

- Assumptions for the "artificial" storage node in the original model (1974) are shown in Figure 3.

- Subsequent modifications of "surcharge computation" applied the same concept, assuming different shapes of the "artifical" storage node. Examples are presented in Figure 4. It followed that various models have predicted different surcharges when tested under the same conditions.

- The most recent version of the model (WRE-IMPSWM, 1979) no longer assumes a surge tank analogy for surcharge computations, and a modified Hardy-Cross method is applied instead as described below.

During surcharge, the continuity equation for node j at time t is

$$\Sigma Q\ (t) = 0 \qquad \text{(V-11a)}$$

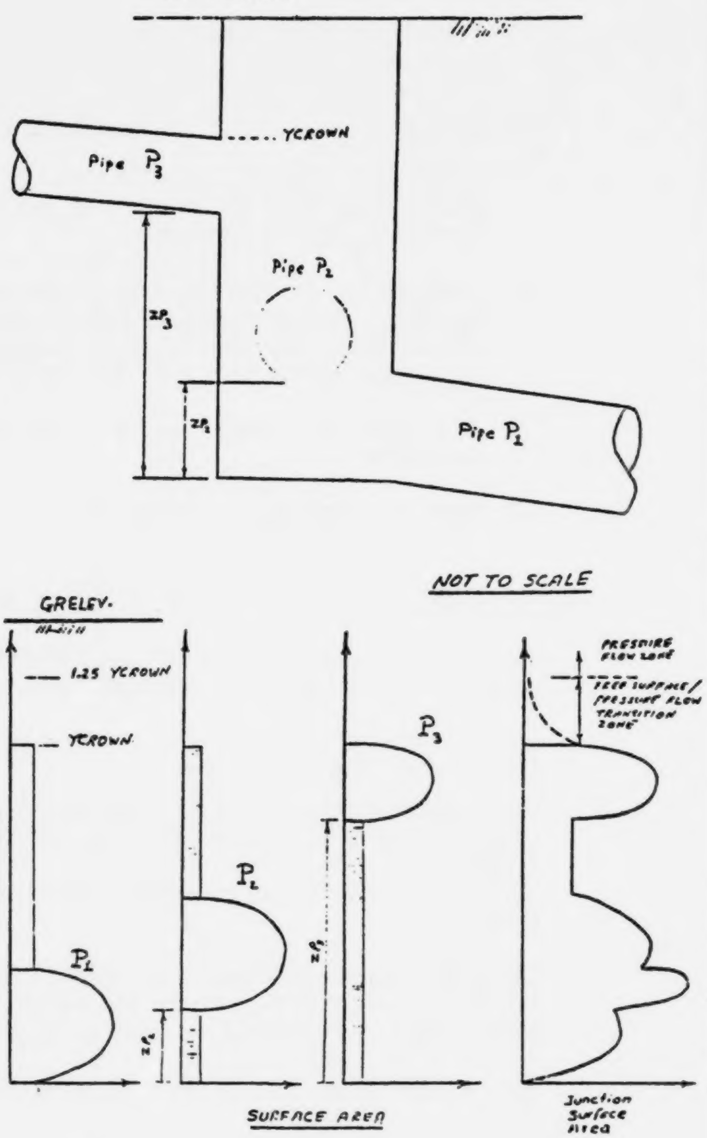

GROUND ELEV.
YCROWN
Pipe P_3
Pipe P_2
ZP_3
ZP_1
Pipe P_1
NOT TO SCALE
GRELEV.
1.25 YCROWN
YCROWN
WATER DEPTH
P_1
P_2
ZP_2
P_3
ZP_3
PRESSURE FLOW ZONE
FREE SURFACE/PRESSURE FLOW TRANSITION ZONE
SURFACE AREA
Junction Surface Area

FIGURE 2 - JUNCTION WATER DEPTH VS. SURFACE AREA AS COMPUTED IN THE WRE-IMPSWM TRANSPORT MODEL (1979 VERSION)

Examples of Storage Assumptions

( ) [illegible] (1975).

[illegible]

The artificial storage node under surcharge is assumed as follows (see Figure 7):

h_o = GRELEV - ZCROWN

h_1 = 0.75 h_o

A_{S_0} = surface area of node at beginning of surcharge - Computed from surface width at 0.96 of the diameter of the uppermost pipe connecting with surcharged node

A_{s_1} = min (5.0/Δt, 0.5 A_{S_0}/Δt)

h_2 = water depth above crown elevation

A_{S_2} = computed surface area of surcharged node corresponding to h_2

The new nodal water depth is computed as follows:

$$Y_2 = Y_1 + (\Sigma Q \times \Delta t)/A_{S_2} \qquad (1)$$

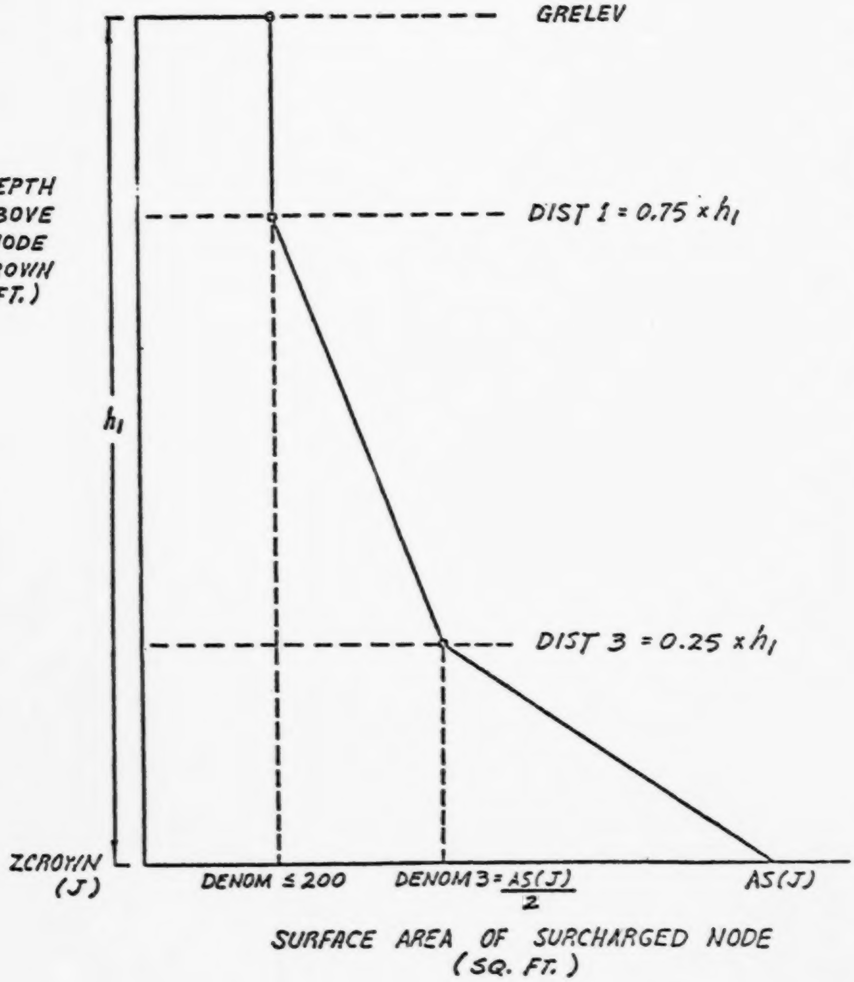
where: Y_2 = new nodal water depth at end of Δt

Y_1 = nodal water depth at start of Δt

ΣQ = summation of flows into junction

FIGURE 3

FIGURE 4 { From Templeton Engineering Company)



REDUCTION OF AS(J) UNDER SURCHARGE IN SWMM 77 where $\Sigma Q(t)$ is all inflows to and outflows from the node from inface runoff, conduits, diversion structures, pump, and outfalls.

Since the flow and continuity are not solved simultaneously in Transport, the flows computed in the links connected to node j will not satisfy equation V-11a. However, computing $\frac{\partial Q}{\partial H_j}$ for each link connected to node j, a head adjustment can be computed such that the continuity equation is satisfied. Rewriting equation V-11a in terms of the adjusted head gives:

$$\Sigma(Q(t) + \frac{\partial Q(t)}{\partial H_j} \Delta H_j(t)) = 0 \qquad \text{(V-11b)}$$

which can be solved for ΔH_j as

$$\Delta H_j(t) = -\Sigma Q(t)/\Sigma \frac{\partial Q(t)}{\partial H_j} \qquad \text{(V-11c)}$$

This adjustment is made by half-steps during surcharge so that the half-step correction is given as:

$$H_j\ (t + \frac{\Delta t}{2}) = H_j(t) + k \Delta H_j(t + \frac{\Delta t}{2}) \qquad \text{(V-11d)}$$

where $\Delta H_j(t + \frac{\Delta t}{2})$ is given by equation V-11c, while the full-step head is computed as:

$$H_j(t + \Delta t) = H_j(t + \frac{\Delta t}{2}) + k\ \Delta H_j(t) \qquad \text{(V-11e)}$$

where $\Delta H_j(t)$ is described by equation V-11c. The value of the constant k theoretically should be 1.0. However, it has been found that equations V-11d and e tend to overcorrect the head; therefore, a value of 0.5 is used for k which gives much better results.

For the various types of links connected to a node, ∂Q/∂H is computed as follows:

Conduits

$$\frac{\partial Q(t)}{\partial H_j} = \frac{32.2}{1-K(t)} \; \Delta t \; \left(\frac{A(t)}{L}\right) \qquad \text{(V-11f)}$$

where

$K(t) = -\Delta t \; \frac{32.2 \; n^2}{2.208 \; R^{4/3}} \; |v(t)|$

Δt = time interval

$A(t)$ = flow cross sectional area in the conduit

L = conduit length

n = Manning n

R = hydraulic radius for the full conduit

$v(t)$ = velocity in the conduit

System Inflows

$$\frac{\partial Q(t)}{\partial H_j} = 0 \qquad \text{(V-11g)}$$

Orifice, Weir, Pump, or Outfall Diversions

$$\frac{\partial Q(t)}{\partial H_j} = (Q(t - \frac{\Delta t}{2}) - Q(t - \Delta t))/(H_j(t - \frac{\Delta t}{2}) - H_j(t - \Delta t)) \qquad \text{(V-11h)}$$

Because the head adjustments computed in equations V-11d and e are approximations, the computed head has a tendency to "bounce" up and down when the conduit first surcharges. This bounding can cause the solution to go unstable in some cases, therefore, a transition function is used to smooth the changeover from head computation by equations V-7 and 8 to equations V-11d and e. The transition function used is:

$$\Delta H_j(t) = \frac{\Sigma Q(t)}{DENOM} \qquad \text{(V-11i)}$$

where

DENOM is given by

$$DENOM = \frac{\partial Q(t)}{\partial H_j} + (AS_j(t) - \frac{\partial Q(t)}{\partial H_j}) \exp(- \frac{15(y_j - D_j)}{D_j})$$

and

AS = the nodal surface area at 0.96 full depth

D_j = pipe diamater

y_j = water depth.

The exponential function causes equation (V-11i) to converge within 2 percent of equation V-11c by the time the water depth is 1.25 times the full flow depth.

CSO IMPACT DETERMINATION
BY LONG TERM SIMULATION

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ROCHESTER, NEW YORK

PRESENTED AT SWMM USERS GROUP MEETING
JAN. 1980
GAINESVILLE, FLORIDA

STUDY SCOPE

GENERAL

The County of Chemung and City of Elmira, New York have to upgrade the treatment of their wastewaters in order to further preserve the Chemung River for its intended best usages.

For the normal wastewater flows (excluding overflows) the most cost effective approach is the construction of a single upgraded facility at the site of the existing City of Elmira Sewage Treatment Plant.

The overflows, of which the city sewer system has two major and three minor, will under present plans continue to operate, pending further study as to their impact. Originally the effect of the overflows was to be studied and analyzed as part of the Southern Tier Central Regional Planning Board's 208 study. Due to financial constraints this aspect of the 208 study was cancelled.

Proper planning and design coordination, however, dictate that the synergistic and complimentary aspects of solutions for wet weather flows and solutions for dry weather flows be examined before finalizing the design of the upgraded regional wastewater treatment plant. Areas to be studied are:

- What effect will increases in plant treatment efficiency have on a needed reduction in overflows.

- What effect will increases in system storage of overflows, with subsequent treatment at the dry weather facility have on project costs and plant operation (dual use of facilities).

- How often do combined discharges of dry weather and treated or untreated overflows contravene stream standards, and for what durations.

To account for the above, the present 201 facility planning study incorporated elements of the 208 planning study and sought to undertake the necessary analyses. These analyses would be of a planning level, and would focus on those combined sewer overflow abatement alternatives that could affect the design or operation of the wastewater treatment plant. For example, what are the consequences of split flow treatment, whereby, all flow beyond a threshold level receives high rate settling in selected settling tanks, with the flow below the threshold receiving normal treatment. If such split flow could achieve significant wet weather contravention reductions, then it may be cost effective to include construction of the required facilities at what could be a relatively minor cost.

Level of Analysis

Urban water management analysis has been refined by the U.S.E.P.A. R & D program as progressing in basically four (4) levels:

Planning
Design

Level I - Desktop Analyses;
Level II - Simplified Continuous Models;
Level III - Refined Continuous Models; and
Level IV - Sophisticated single event and continuous models

The level at which the subject study is being performed is within Level III. This is at the upper end of the planning analysis spectrum. A study of the City of Elmira's overflow situation at this level was desired because, one, the results could impact on the design of the treatment plant, and two, the results of this study will be used to determine if a CSO problem exists to what severity, and to formulate a continued program.

To accomplish the above, knowledge of the overflows and receiving waters during transient storm events is required. The scope of the problem then has determined the need of accounting for the quantity and pollutional content of the overflows for dry and wet weather conditions. Complete dependence on empirical measurements is not desirable; the time requirements and number of monitoring stations would be prohibitive. What was needed was a basis of analysis which augmented available data by incorporating empirical measurements, and generalizations derived from other systems and from theory, to calculate historical overflow quantity and quality.

NEEDS OF ANALYSIS

REQUIREMENTS OF ANALYSIS

The magnitude and comprehensiveness of the study required the use of computerized mathematical modeling and associated software to make it possible to process the necessary information in a reasonable time period and with useful accuracy. The basic requirements to be satisfied by the methodology were:

- To account for the time variations in precipitation, runoff, dry periods, and receiving water conditions by a continuous multiple event analysis.
- To have the model initialize each event.
- To provide statistical analysis of the output data.
- To simulate the flows and pollutant characteristics in sufficiently short time intervals to adequately represent varying flow rates and receiving water reactions.

NEED FOR MULTIPLE EVENT ANALYSIS

Urban stormwater runoff and stream flows are determined by a randomly distributed hydrological process wherein rainfall, runoff, and overflow, and impacts on receiving waters each have differing frequencies of occurrence. Meaningful and realistic assessments concerning urban runoff pollution, intermittant loadings of sewer system overflows and receiving water impacts are not readily obtained from simulation of singular events. The numerous, highly interdependent influences and especially the random nature of storm events and stochastic nature of streamflows necessitate the investigation of many events, to gain the proper perspective of the entire process.

A basic consideration is that the impact of overflows on river water quality is highly event-specific. Analyses based on total annual removal of overflow, or on a percent removal from a design event, have little meaning. Overflow characteristics are clearly dependent on storm intensity for surface wash-off and sewer scour, quantity of dust and dirt on surfaces and deposition in sewers, accumulated pollutant wash-off during the process of a storm, and the filling of surface depressions during the progress of a storm. The river's assimilative capacity is dependent upon flow, temperature, photosynthetic activity, etc.

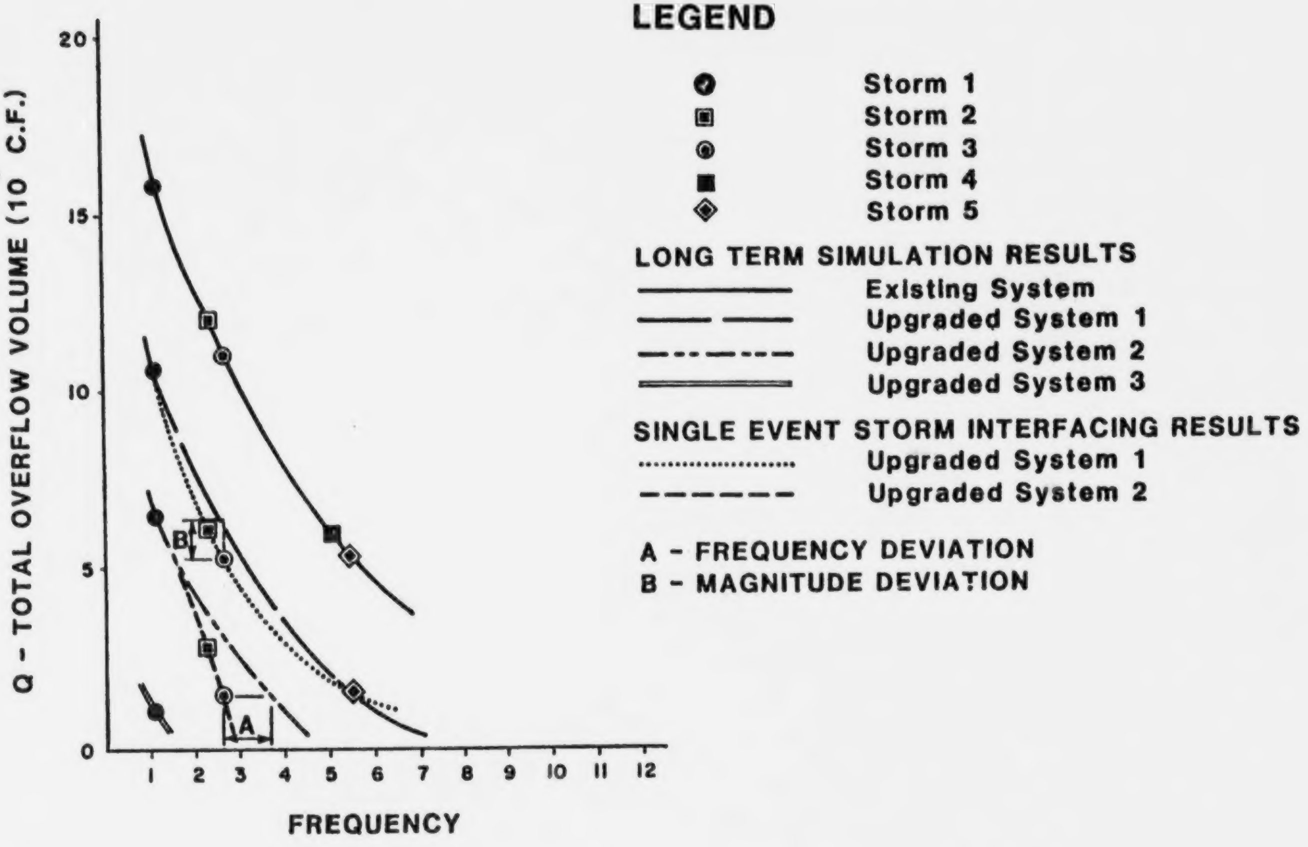
It is interesting to note that for a given storm event the respective frequencies of rainfall, runoff, overflow quantity, overflow quality, and, river assimilative capacity may all differ, making it difficult to fabricate a single, or even a set of representative "design" storms.

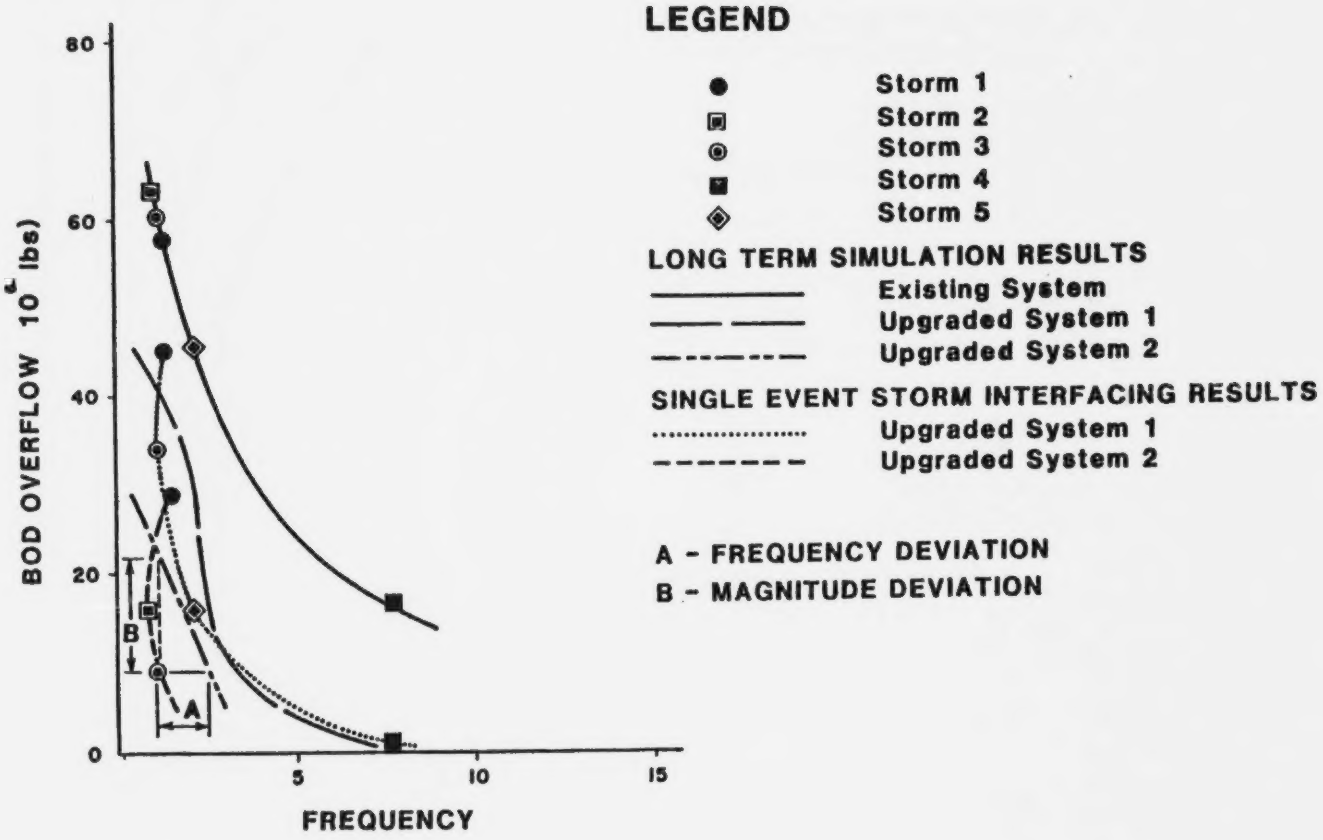
SINGLE EVENT SIMULATIONS

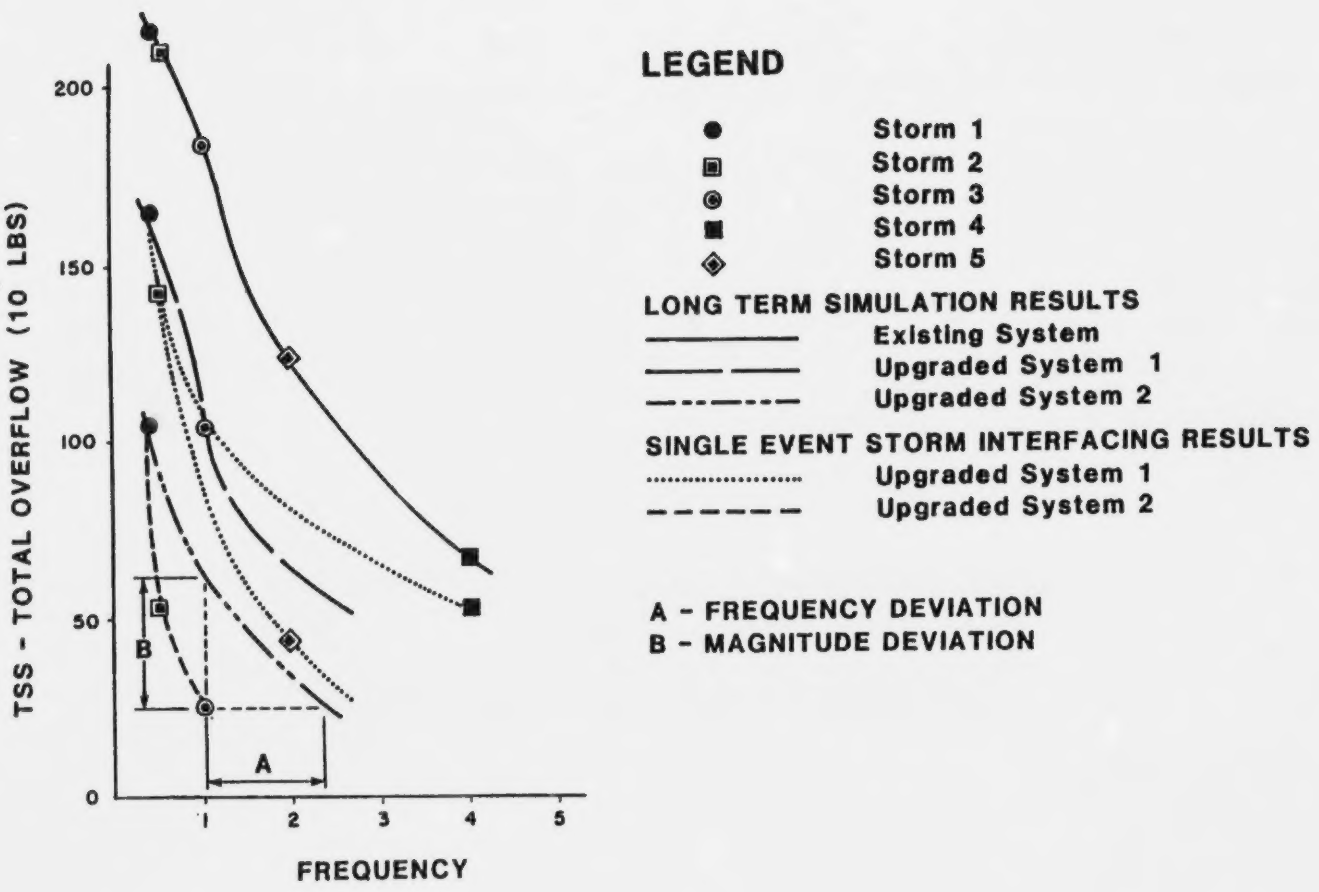
During a prior study the feasibility and practicality of benchmarking single event storms with long-term analyses for purposes of reducing the work effort on future analyses was examined. The results of the analysis showed that such a technique could be erroneous and misleading.

The benchmarking study was performed using long-term continuous simulation output for the parameters of flow, BOD, and TSS. The total quantity discharged of each parameter from storages of several select sizes for several select real time single-events were compared. The primary thesis employed in the benchmarking attempts was that the frequency associated with each event on the existing system, would not shift or would shift only slightly, while the actual quantities of flow, BOD, and TSS would change for each size of storage improvement. The test was performed by varying the size and outlet flow of the storage, with each size being considered as a different level of improvement. For each level of improvement a frequency curve was developed (Figures A, B, C & D). The selected individual storm results from each improved system were then benchmarked against the frequency curve, developed from continuous simulation, for the existing system. The frequency curve generated through the benchmarking procedure was then compared with the frequency curve based on the long-term analysis.

As it developed, for the method of analysis employed, the thesis proved false. The parameter most closely approximating the primary thesis was that of total overflow quantity. Even here though, the deviations are significant, to the point that confidence in the procedure is lacking.


Q - TOTAL OVERFLOW VOLUME (10^6 C.F.)
0
5
10
15
20
FREQUENCY
1
2
3
4
5
6
7
8
9
10
11
12
A
B
LEGEND
● Storm 1
▣ Storm 2
◉ Storm 3
■ Storm 4
◈ Storm 5
LONG TERM SIMULATION RESULTS
Existing System
Upgraded System 1
Upgraded System 2
Upgraded System 3
SINGLE EVENT STORM INTERFACING RESULTS
Upgraded System 1
Upgraded System 2
A - FREQUENCY DEVIATION
B - MAGNITUDE DEVIATION


BOD OVERFLOW 10^6 lbs)
80
60
40
20
0
5
10
15
FREQUENCY
A
B
LEGEND
● Storm 1
▣ Storm 2
◎ Storm 3
■ Storm 4
◈ Storm 5
LONG TERM SIMULATION RESULTS
Existing System
Upgraded System 1
Upgraded System 2
SINGLE EVENT STORM INTERFACING RESULTS
Upgraded System 1
Upgraded System 2
A - FREQUENCY DEVIATION
B - MAGNITUDE DEVIATION


TSS - TOTAL OVERFLOW (10^3 LBS)
200
150
100
50
0
1
2
3
4
5
FREQUENCY
A
B
LEGEND
● Storm 1
▣ Storm 2
◎ Storm 3
■ Storm 4
◈ Storm 5
LONG TERM SIMULATION RESULTS
Existing System
Upgraded System 1
Upgraded System 2
SINGLE EVENT STORM INTERFACING RESULTS
Upgraded System 1
Upgraded System 2
A - FREQUENCY DEVIATION
B - MAGNITUDE DEVIATION

FREQUENCY OF OCCURRENCE

(times/year)

(As Determined From Long -Term Simulation)

STORM EVENT No. 3

	Existing	1/5 Yr. Design	1/2 Yr. Design
Overflow Q (cf)	2.4	3.1	3.7
BOD (lbs)	1.0	1.6	2.5
TSS (lbs)	1.0	1.1	2.4

STORM EVENT No. 4

	Existing	1/5 Yr. Design	1/2 Yr. Design
Overflow	5.0	7.0	No Overflow
BOD	7.7	7.0	
TSS	4.0	2.5	

A further analysis was performed in a similar manner, using a different set of single-event storms chosen from the same modeled years of record; selected from events having overflow/frequency relationships approximating the first set.

The interfaced frequency curve generated from the second set of storms not only failed to agree with the long-term frequency curve, but also failed to agree with the interfaced frequency curve generated from the first set of storms.

The failure of the tested procedure to demonstrate adequate generation of frequency relationships for the improved systems is understandable. The model employed for the study considered, among other items, input parameters such as storm intensity for surface wash-off and sewer scour, antecedent dry period for pollutant wash-off during the progress of a storm, and the filling of surface depressions during the progress of a storm. The difficulty in benchmarking individual storms further demonstrates the need for an approach other than by "design storm".

NEED FOR CONTINUOUS SIMULATION

The build up of surface contaminations, the build up of settled matter in the sewer network, the emptying of storage, and the drying of surface areas are dependent upon the period between storms. Each of these parameters affects the runoff, or the pollution potential, or both. In turn each of these parameters is affected by the intensity and duration of rainfall. For example, there might have been only partial washoff from a previous storm. For reasonable assessments of overflow quantity, duration, and quality, simulated for each storm event, it is necessary that these parameters be initialized. To initialize these parameters for all components for each event is quite time consuming, and introduces another source of possible error in the analysis. It is desirable to have the means to trace the conditions through a continous simulation.

NEED FOR DYNAMIC SIMULATION

The overflow process is closely related to the hydraulic capacity and the actual flows within the sewer network which it relieves. Similarly the receiving waters behavior is closely related to its hydraulic capacities and actual flows. In simulating overflow occurrences, it is then important to represent the unsteady flow rates in the network and receiving waters with reasonable accuracy. This requirement implies sufficiently short time intervals of simulations to correspond with intermittent conditions in the system.

In previous studies on the Rochester, New York system, a computerized simple mass balance computation on long-term rainfall data was accomplished to test the difference using daily and hourly rainfall input into a model. The analysis indicated that an hourly data input would produce a sixty (60) percent increase in overflow quantities over that produced by daily input data. Initial calibrations of the storm model on this study indicated that by using hourly increments reasonable results in matching peak and total flows can be obtained. Given the above and the nature of stream hydraulics, including initial mixing diffusion, backmixing, and the relative speed of the quality reactions, one (1) hour time intervals were selected.

NEED FOR STATISTICAL APPROACH

Rainfall, runoff, overflow and streamflow are all probabilistic or stochastic parameters, and should be analyzed as such. With the amount of data to be analyzed, the only practical approach is a statistical one, incorporating records of many events into a picture of the network as it operates under varying conditions.

A statistical approach offers two definite additional advantages. One, the model is only an approximation of the actual network and deviations from reality exist in all parameters and all internal functions. By operating the model over many simulated storm events under varying conditions and then statistically analyzing the data, the effect of errors associated with an individual event is minimized. Two, the receiving water standards are open to interpretation from a frequency of contravention standpoint. A statistical analysis fits nicely with this interpretation.

Model Selection

As a summary of the previous discussion, a model package to be selected had to meet the following criteria:

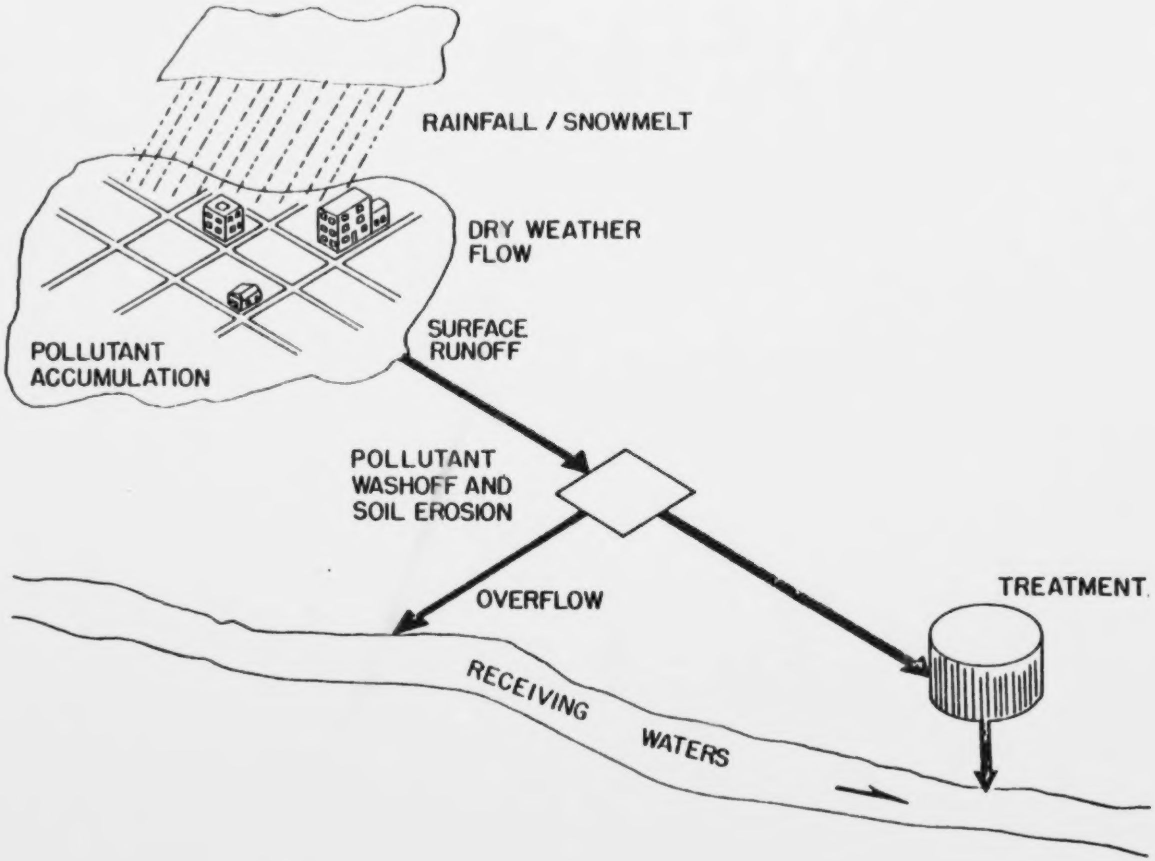
- Be suitable for planning level purposes;
- Dynamically and continuously analyze the overflow process;
- Dynamically and continuously analyze the effects of the overflows on the receiving waters; and
- Statistically analyze the output.

The model package seen to best meet the above criteria was the: U.S. Army Corps of Engineers Hydrologic Engineering Center's (HEC) Storage, Treatment and Overflow Reduction Model (STORM); the HEC Water Quality River Reservoir Systems (WQRRS), and the HEC statistical post processor program.

THE STORM MODELING CONCEPT

Storm is a continuous simulation model used for prediction of the quantity and quality of storm water and dry weather flow. The model provides analysis that satisfies two primary study objectives. These are: (1) formulation of wet weather hydrographs and pollutographs for use in receiving water impact assessment and (2) provide statistical information to aid in the selection of storage capacities and treatment rates to achieve a given level of control of storm water runoff. The approach used in the model to achieve these objectives is the recognition of the properties of storm duration and intensity, the storage capacity of the system and the affects of storm event spacing.

The overall model operation involves the interaction of 8 main processes. These are: precipitation, runoff, pollutant accumulation, pollutant washoff, dry weather flow, storage, treatment and overflow. Figure 1 shows a schematic representation of the 8 storm water elements modelled. The model computes runoff from rainfall or rainfall plus snow melt and associated pollutant washoff for a lumped basin. In this process, rainfall washes dust and dirt and associated pollutants off the watershed, scour of debris in the conveyance system is not directly modelled as such but can be accounted for by an appropriate calibration technique. Runoff in excess of a specified treatment rate is diverted into storage for subsequent treatment. Runoff in excess of both the treatment rate and storage capacity is considered overflow and is diverted directly into the receiving waters.


RAINFALL / SNOWMELT
DRY WEATHER FLOW
POLLUTANT ACCUMULATION
SURFACE RUNOFF
POLLUTANT WASHOFF AND SOIL EROSION
OVERFLOW
TREATMENT.
RECEIVING WATERS

MAJOR PROCESSES MODELLED BY STORM

Figure 1

The quantity, quality and number of overflow events are thus functions of hydrologic characteristics, land use, treatment rate and storage capacity. In keeping with this there are three major steps involved in estimating storm water runoff and quantity and quality, these are: 1) the computation of runoff quantity, 2) the computation of runoff quality, and 3) the computation of treatment, storage and overflow.

COMPUTATION OF RUNOFF QUANTITY

Runoff quantity can be computed by one of three methods, the coefficient method, the U.S. Soil Conservation Service Curve Number Technique or a combination of the two. For a predominately impervious lumped urban catchment the coefficient method is selected based on the relative ease of calibration and the relative significance of the previous area. Storm employs the Soil Conservation Service triangular unit hydrograph as a means of routing basin excess to the point of treatment/storage/overflow for both the coefficient method and the SCS curve number technique.

The coefficient method defines runoff as the product of a runoff coefficient and hourly rainfall excess. The runoff coefficient is the weighted average of empirical runoff coefficients for pervious and impervious areas, as such it is a composite runoff coefficient which accounts for infiltration losses. The rainfall excess is defined as the difference between hourly rainfall and losses to depression storage. The composite runoff coefficient is used for every rainfall in the rainfall/snow melt record regardless of rainfall characteristics or antecedent moisture conditions. Antecendent depression storage is defined as the available depression storage at the end of the pervious rainstorm plus a linear recovery to account for evaporation during the period of no precipitation. The evaporation rate can be verified monthly. The coefficient method is a simplification of the hydrology, but it can be expected to perform relatively well on an urban catchment of relatively high percent imperviousness for which losses due to infiltration are relatively small. The runoff coefficients, available depression storage and depression storage recovery factor are derived in the calibration procedure using observed data.

COMPUTATION OF RUNOFF QUALITY

Computations for the stormwater runoff quality are based on formulations first used in the EPA Stormwater Management Model. Empirical equations considering land use, street sweeping practices, and days between rainstorms define the amount of each pollutant on the ground at the beginning of the rainstorm. An exponential washoff equation relates the mass of pollutants washed off during each hour to the current mass of pollutants on the watershed, the runoff rate and an exponent governing the rate of pollutant washoff.

Two methods are available in STORM for the determination of pollutant accumulation. These are the dust and dirt method and the daily pollutant accumulation method, the latter is not intended for use on an urban catchment. The dust and dust method assumes that all pollutants are associated with the dust and dirt accumulation in the streets. This assumption is acceptable for storm sewers and open channel conveyance system but is not a proper characterization of a combined sanitary and storm water drainage system. In a combined sewer system there is typically a build-up in the sewers of grit and organic debris between rainstorms. Thus, in addition to the surface accumulation of dust and dirt there is the addition pollutant load flushed from the conveyance system, i.e. the conveyance system scour and washout.

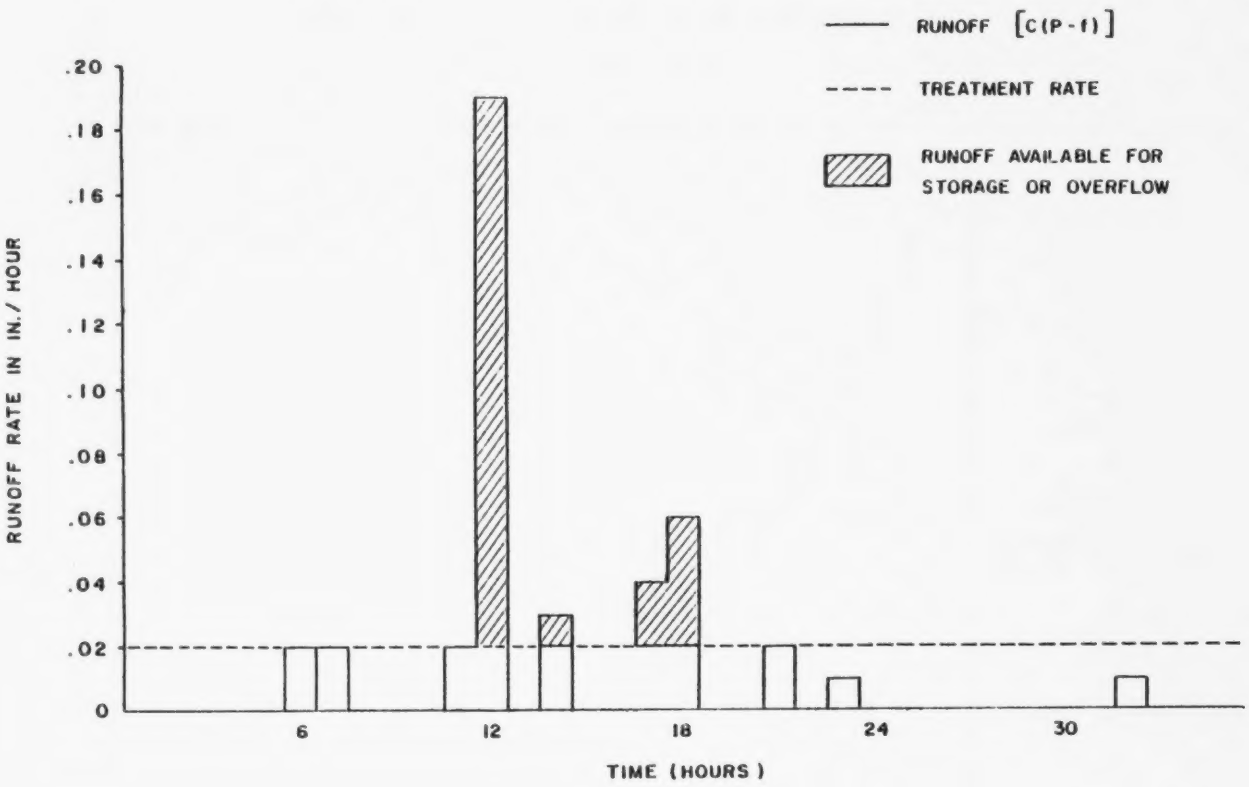
Calibration of the dust and dirt method for separate stormwater conveyance systems involves adjustment of the dust and dirt accumulation rates, the pollutant fractions of dust and dirt and the washoff coefficient so that the predicted pollutant concentrations most nearly match those from measured data. When calibrating the dust and dirt method for combined storm and sanitary conveyance systems the system washout must be lumped in with the surface dust and dirt. A combined sewer overflow sampling program is required when calibrating STORM for a combined Storm/Sanitary conveyance system. The quality constituents predicted by the surface runoff portion of STORM are suspended solids, settleable solids, biochemical oxygen demand, total nitrogen, total orthophosphate and total coliform. Each constituent is determined for an hourly computation interval. The computation interval affects the lower limit of size of catchment that may be modelled. As normally used, STORM should not be expected to produce properly shaped hydrographs or pollutographs for catchments having times of concentration less than one hour.

The quantity and quality of dry weather flows are input to STORM as average values for domestic, commercial, industrial and infiltration loads. They are modified on a daily and hourly basis by a user specified coefficient.

The dry weather flow and quality hourly hydrograph and pollutograph is added to those of the runoff and total hydrographs and pollutographs are generated.

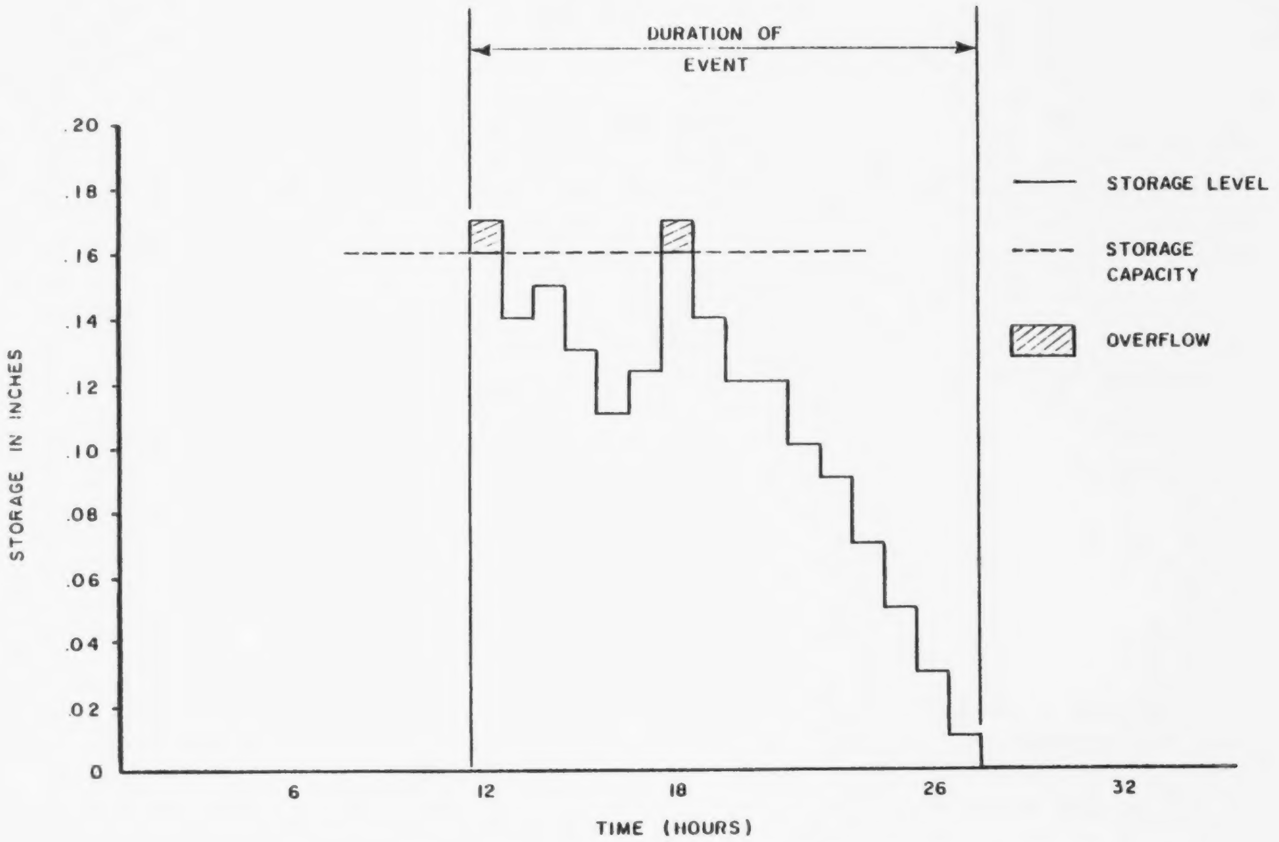
STORAGE, TREATMENT AND OVERFLOW

Computation of treatment, storage, and overflow proceeds on an hourly basis throughout the rainfall snowmelt record. Periods of no rain are skipped, however, the number of dry hours is used for various purposes including build-up of surface dust and dirt. Each hour in which runoff occurs the treatment facilities are utilized to treat as much runoff as possible. When runoff rates exceed the treatment rate, storage is utilized to contain the runoff. When runoff is less than the treatment rate, the excess treatment rate is utilized to diminish the storage level. If the storage level is exceeded, all excess runoff is considered overflow and does not pass through the storage facility. This



TIME HISTORY OF RUNOFF USING THE COEFFICIENT METHOD

Figure 2



TIME HISTORY OF STORAGE USING THE COEFFICIENT METHOD

Figure 3

overflow from the system becomes the input hydrograph and pollutograph for a receiving water quality model. Plug flow is assumed for routing of pollutants through storage, water quality is not modified during the storage routing interval.

Figures 2 and 3 show the time histories of run-off, storage and overflow. From the time histories it can be seen that for a given planning horizon, overflows to a receiving water could be minimized by examining a matrix of land use alternatives and treatment rate/storage alternatives.

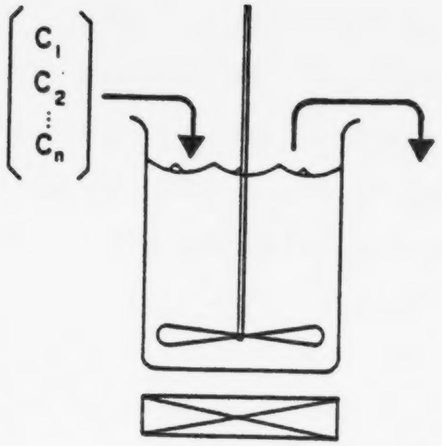
Logically, the planning horizon should consider the impact of overflows from the existing urban catchment on the receiving water. Based on the overflow impact of the existing system a matrix of alternatives land use and treatment/storage systems can be designed to determine the best cost benefit/cost effective solution to the CSO problems.

LOZIER ENHANCEMENTS TO STORM

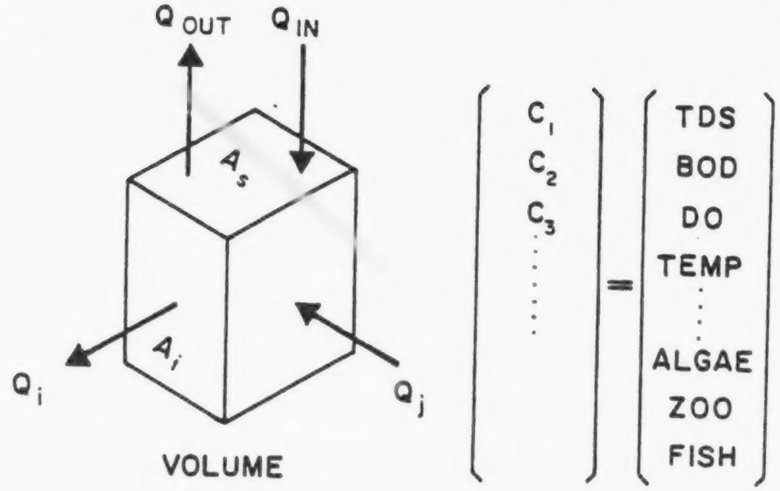
Several program enhancements have been developed into the original HEC version of STORM by Lozier. These program changes were developed to enable the model to more realistically represent the operational aspects of the storage/treatment system. It was recognized by Lozier that at the end of a rainfall/runoff event (the end of excess rainfall) wastewater remaining in storage was lost to the analysis. The model was altered to permit the event to continue until all storage was bled back through the treatment works resulting in the time extension of the pollutograph output. Additionally, the option was provided to permit the user to specify the treatment efficiency for reduction of the six modelled quality constituents. A different treatment efficiency can be assigned to each quality constituent. This permits the user to examine the impact on the receiving water of treated effluent and stormwater overflow as the treatment works is pushed hydraulically during the period of wet weather treatment. This option can also be used to add a dimension to the matrix analysis of land use/storage/ treatment by permitting the comparison of different treatment processes at various levels of hydraulic loading.

THE WORRS MODELLING CONCEPT

The Water Quality River-Reservoir modelling concept is based on the fundamental principal of the Law of Conservation of Mass and the Kinetic Principle, as they apply to studies of the Continually Stirred Tank Reactors (CSTR). Laboratory studies are largely responsible for the current level of understanding of each individual process that occurs in a receiving water environment. The problem of receiving water modelling is to assemble the body of theoretical and emperical knowledge developed from CSTR experiments into a simulator that properly predicts the phenomena observed in the field, i.e., the prototype. Assembling such a simulator is not a simple task because CSTR studies are normally conducted at relatively small scale usually under well controlled conditions and often under different circumstances than those existing in a prototype. The receiving water is not a single simple reactor, to overcome the disparity between model and prototype the receiving water is conceptualized as a system of discrete hydraulic or volume elements.



A. A CONTINUOUSLY STIRRED TANK REACTOR, CSTR

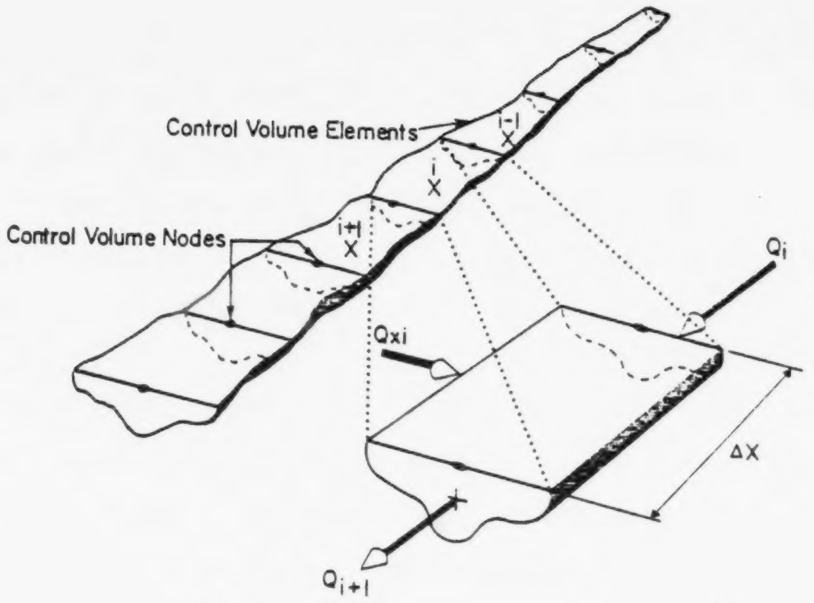

Q OUT
Q IN
A s
A i
Q i
Q j
VOLUME
C1
C2
C3
=
TDS
BOD
DO
TEMP
ALGAE
ZOO
FISH

B. AN IDEALIZED HYDRAULIC ELEMENT

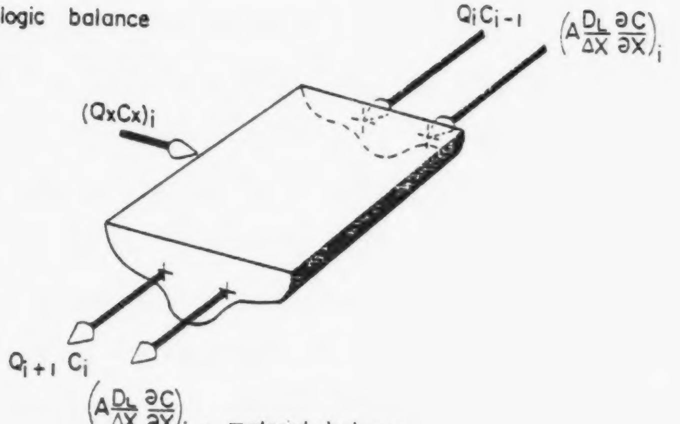
Figure 4

Figure 4 represents an idealized hydraulic element where the state of the ecosystem can be defined and where the CSTR analogy holds. The element has a volume, a surface area and one or several cross-sectional areas between adjacent elements. Flows are advected from adjacent elements and local inflows and outflows are accounted for. The conceptualization is carried one step further by inter-connecting the hydraulic elements in such a way that they can be viewed as a series of CSTR's whose contents can be transferred from one to another. This analogy improves as the number of hydraulic elements is increased. Figure 5 illustrates the hydraulic element placed in series to represent stream system geometry and mass transport mechanism.

The mechanics of receiving waters are usually highly dynamic. The CSTR on the otherhand usually represents steady state conditions. Consequently, the CSTR analogy can only be considered valid when the prototype approximates steady state conditions. To overcome this disparity the CSTR or hydraulic element must be observed at a sufficiently frequent interval that will permit the characterization of the dynamic process as a set of successive steady state processes. As the observation interval or time step is shortened the CSTR/hydraulic element analogy improves. Many reactions are occurring simultaneously in the receiving stream. There is no theoretical difficulty in dealing with this if in the mathematical simulation the reactions are considered to properly take place in parallel (independent reactions) or in series (inter-dependent reactions). Each process contributing


Control Volume Elements
Control Volume Nodes
i-1
X
i
X
i+1
X
Q_i
Q_{xi}
ΔX
Q_{i+1}

hydrologic balance


Q_iC_{i-1}
$\left(A\frac{D_L}{\Delta X}\frac{\partial C}{\partial X}\right)_i$
$(Q_xC_x)_i$
$Q_{i+1}\ C_i$
$\left(A\frac{D_L}{\Delta X}\frac{\partial C}{\partial X}\right)_{i+1}$

material balance

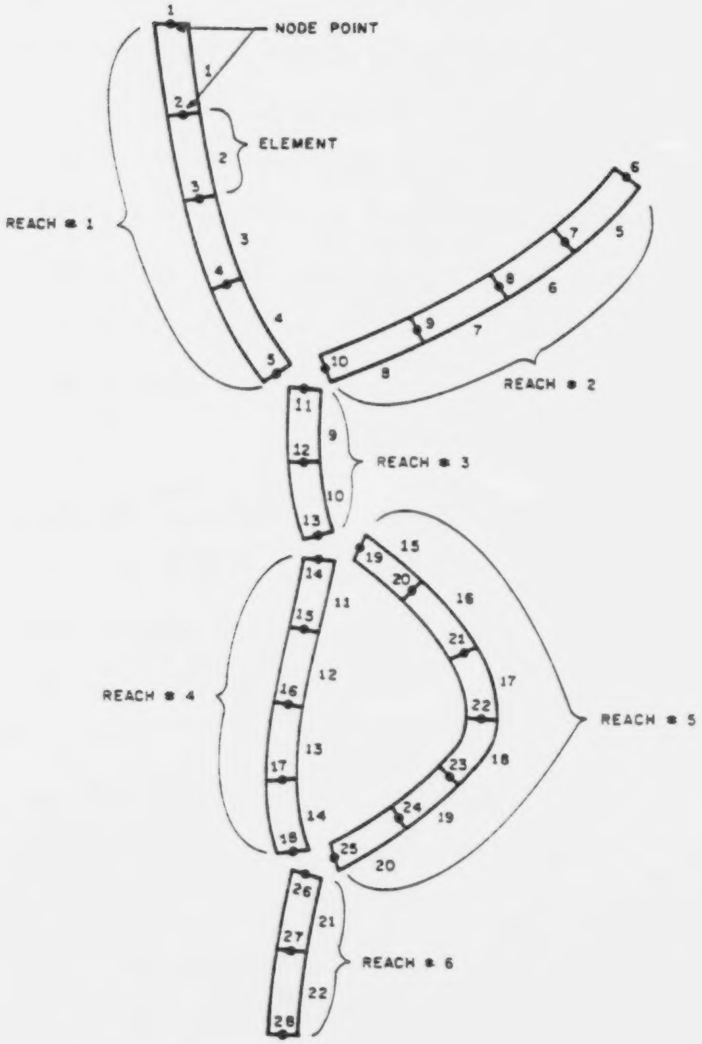
DISCRETIZED STREAM SYSTEM

Figure 5

in part to the total water quality change is mathematically represented by a partial differential according to kinetic principles. Partial differentials can be added together to form the total differential, allowing the computation of total response in the water quality. Therefore, for each hydraulic element a mass balance equation in differential form is solved for each water quality constituent. The whole array of differential equations representing the coupled nature of the water quality constituent is entered for each time step and solved to determine the dynamic response of the receiving water to wasteloads.

WQRRS Model Structure

The WQRRS model consists of three separate but integrable modules; the reservoir module, the stream hydraulics module and the stream quality module. WQSTAT, a statistical and graphical package, is available for reduction of the stream quality module output. The reservoir and system hydraulic module are stand alone programs and may be executed analyzed and interperted independently. The stream quality module has no hydraulic computation capability and requires a hydraulic data file generated by the stream hydraulics module. The three computer programs can be integrated for a complete river basin water quality analysis through the automatic storage of result for input to downstream simulations.


NODE POINT
ELEMENT
REACH # 1
REACH # 2
REACH # 3
REACH # 4
REACH # 5
REACH # 6
1
2
3
4
5
6
7
8
9
10
11
12
13
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28

REPRESENTATION MODEL
TYPICAL STREAM SYSTEM

Figure 6

STREAM HYDRAULICS MODULE

The stream system is represented conceptually as a linear network of hydraulic elements. Several elements make up a stream reach, several reaches comprise a stream system. Figure 6 illustrates a typical stream system representation, note that branch and looped flow can be modeled. Each element is characterized by length, width, cross section, and certain other parameters such as location of tributaries and local inflows and outflows. Six methods of hydraulic computation are incorporated into the hydraulic module. They include:

1. Backwater hydraulic solution (steady flow)
2. Solution of the full St. Venant equations
3. Solution of the kinematic wave equations
4. Direct input of a stage-flow relationship (steady flow)
5. Muskingum hydrologic routing
6. Modified puls hydrologic routing

The first three methods represent hydraulic behavior of the prototype stream system under gradually varied flow and assumes the system can be represented by the St. Venant equation of motion. Additional assumption for the first three methods are:

1.) the system is one dimensional in a mathmatical sense, flow and velocity are uniform both laterally and vertically

2.) linear interpolation between cross-sections provides adequate definition of the element system.

3.) The rate of energy loss for gradually varied steady and unsteady flow is the same as that for uniform flow having the same velocity.

4.) The slope of the channel bottom is small (ie., cos $\theta = 1$)

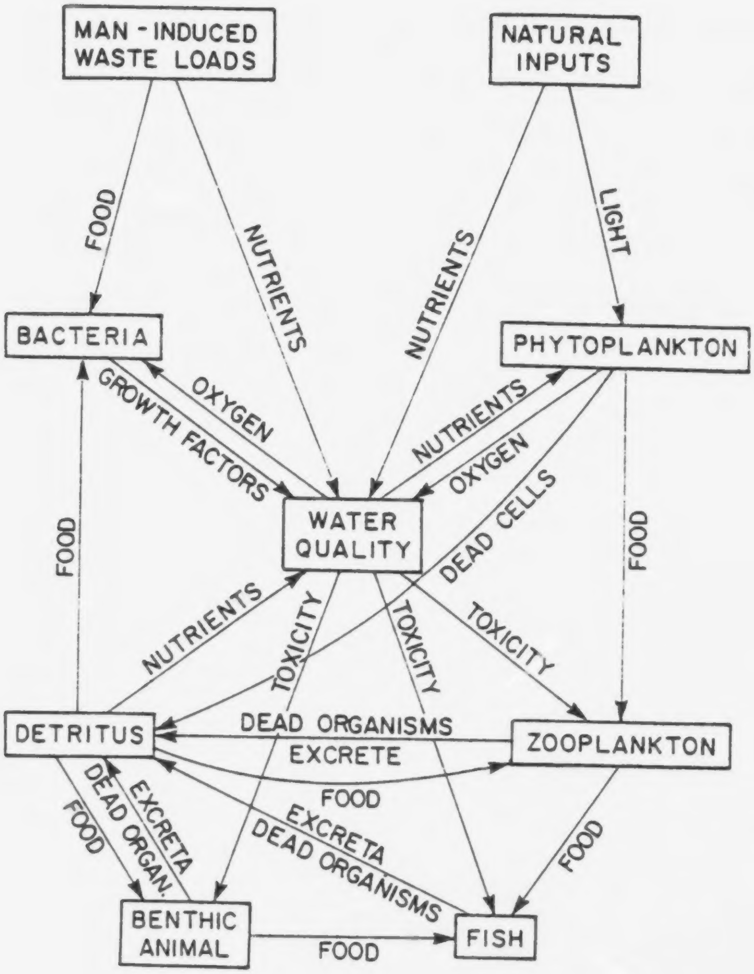
The final three hydraulic computation methods are all independent of the fundamental hydraulic relationships as represented by the St. Venant equation. Each method requires specific boundary conditions of flow, stage, or flow versus stage.

Input to the hydraulics module consist of a table of elevation versus channel characteristics for each element. The table of characteristics consist of cross-section area, width, hydraulic radius, and friction factor. This input can be generated by the use of the HEC program, Geometric Element From Cross-Section Data (GEDA). GEDA processes available cross section data to produce the elevation versus channel characteristics table at desired locations. The hydraulic module processes inflow and withdrawal records input at user specified locations and time intervals, ie., hourly, daily, monthly. The module output consists of, flow depth, velocity, cross-section area surface area and width for each element at a user specified computational interval. This output is saved on an interface storage devise, such as magnetic tape or disc, for subsequent use in the stream quality model. Thus the hydraulic module output characterize the spatial and temporal relationships of the stream system. This characterization includes base stream flow and tributary flows, inflows such as waste water discharge rate and rainstorm overflow rates and stream flow withdrawals in addition to the hydraulic aspects relevant to element and reach travel time and stream reaeration.

Quality Module

The receiving water quality module is based on the assumption that the dynamics of each chemical and biological water quality component can be expressed by the law of conservation of mass and the kinetic principle. An important assumption is that all chemical and biological rate process occur in an aerobic environment.

The ecological processes within the quality module are centered around benthic alage where the tropic relationship between benthic alage and aquatic insects form the base of the food chain. Figure 1 shows the interrelationship of the food chain constituent and water quality constituents. Figure 7 also illustrates the inter-dependance or coupled nature of the water quality constituents and ecological constituents.


MAN - INDUCED WASTE LOADS
NATURAL INPUTS
FOOD
NUTRIENTS
NUTRIENTS
LIGHT
BACTERIA
PHYTOPLANKTON
OXYGEN
GROWTH FACTORS
NUTRIENTS
OXYGEN
DEAD CELLS
FOOD
WATER QUALITY
FOOD
NUTRIENTS
TOXICITY
TOXICITY
TOXICITY
DETRITUS
DEAD ORGANISMS
EXCRETE
ZOOPLANKTON
FOOD
FOOD
DEAD ORGAN.
EXCRETA
EXCRETA
DEAD ORGANISMS
FOOD
BENTHIC ANIMAL
FOOD
FISH

DEFINITION OF AN AQUATIC ECOSYSTEM

Figure 7

The quality module incorporates a water quality constituent disconnect capability which allows the user to restrict the simulation of constituents to those of interest or importance in a specific study. The user may also hold a constituent at a constant value during the simulation. This permits the user to adapt the model to a particular level of study, which could range from a simple first cut simulation of BOD, dissolved oxygen and temperature or a very rigorous simulation of the aquatic food chain. The importance of this option should be fully considered by the user, as once the modeling data base has been assembled for the stream system, it can be updated as information becomes available and the level of simulation can be upgraded accordingly.

A Gaussian reduction technique is used to solve the set of simultaneous equations representing those constituents which are passively transported with the movement of water. For those constituents which are assumed affixed to the bottom or are self mobile, such as fish, the equations are solved by simply multiplying the time derivatives by the computation time step increment. The differential equations representing the constituents are processed sequentially from the least dynamic to the most dynamic. The array of differential equations is structured such that the coupled nature of the constituent is recognized in the sequential regression.

Water quality constituent sources and sinks resulting from coupling are assumed constant over the computational time step. These sources and sinks may include: settling, first order decay, reaeration, biological uptake and release, growth, respiration, and mortality including predation. With the exception of conservative constituents the differential equations representing water quality relationships incorporate one or more physical, chemical or biological rate coefficients. These rate coefficients are based upon theoretical and empirical understanding of the processes governing the sources and sink terms. Many of these rate coefficients are highly variable and depend upon such factors as regional climatic variation, time of day, synoptic weather patterns, stream system geometry and type and general levels of pollution.

In the aquatic environment the rates at which chemical and biological processes take place are normally functions of temperature, therefore rate coefficients describing these processes must be adjusted to the ambient temperature. Two approaches are used in the model to make the temperature adjustment. The temperature limit method assumes a reaction takes place as a function of two exponental curves constrained by temperature tolerance limits. One curve considers growth, respiration, mortality and decay, the other considers growth only. The temperature correction method assumes the rate at which a reaction takes place increases exponentially with increase in temperature. This method applies to stream reaeration coefficients and coliform die-off rates, remaining constituents can be specified for temperature correction by either method.

The ambient temperature is arrived at in the model by considering external heat sources and sinks. In a stream system heat is transferred at the air water interface and the stream bottom sediments.

Two methods may be selected by the user to calculate the water surface heat flux; the heat budget method and the equilibrium temperature method. Determination of the rate of heat transfer per unit surface area for both methods requires the following five components; net rate of short-wave solar radiation, net rate of atmospheric long-wave radiator, rate of long wave radiation from the water surface, rate of heat loss by evaporation and rate of convective heat exchange between the water surface and the overlying air mass. This information is generated in the model from standard meteorological input consisting of wet and dry bulb temperature, cloud cover, barometric pressure and wind speed in addition to invariant data such as atmospheric turbidity, evaporation coefficients and latitude and longitude. Variant meterological data is imput at a time interval (hours, days, months) consistent with available information and the level of study required.

Heat exchange with bottom sediments may be significant in shallow streams where the heat exchange with the bottom will have a moderating effect on water temperature fluctuations. The maximum daily temperature will be reduced by conducting of heat away from the water to the cooler bottoms sediments. The reverse will occur at night, thus the total temperature fluctuation is moderated.

WQRRS
ANALYTIC PROCESSES

QUALITY MODULE

- THE USER HAS THE OPTION OF SELECTING FROM THE FOLLOWING SEVEN METHODS FOR COMPUTING THE DISSOLVED OXYGEN REAERATION COEFFICIENT:

CHURCHILL	$K_2 = 5.031 \left(\frac{V^{.969}}{H^{1.673}}\right)$
O'CONNOR & DOBBINS	$K_2 = 3.951 \left(\frac{V^{.5}}{H^{1.5}}\right)$
OWENS	$K_2 = 5.346 \left(\frac{V^{.67}}{H^{1.85}}\right)$
LANGBIEN AND DURUM	$K_2 = 5.133 \left(\frac{V}{H^{1.333}}\right)$
THACKSTON & KRENKEL	$K_2 = 24.95 \left[(1+F^{.5})\frac{V^{*}}{H}\right]$
TSIVOGLOU & WALLACE	$K_2 = 3.78 \left(\frac{\Delta h}{\Delta t}\right)$
DIRECT INPUT OF K_2	

* SHEAR VELOCITY = $(H \cdot S \cdot g)^{.5}$

TABLE 1

The response of water quality to external wasteloads is determined by routing the wasteload through the stream system. In the model this takes place after initial conditions have been satisfied for the water quality constituents selected for simulation. During the routing procedure, for each computational interval, the temperature is determined, biological and chemical rate coefficients are adjusted, the stream dissolved oxygen reaeration capacity is determined and the water quality constituent response is predicted. This result becomes the initial condition for the next time step and so on as the water moves downstream. As the water moves through the stream system, the dissolved oxygen concentration can be determined based on the hydraulic characteristics of the stream flow. Table 1 lists the six computational methods available to the user for determining the dissolved oxygen reaeration coefficient. In addition, the user may directly input the reaeration coefficient.

Quality module output consists of a water quality constituent profile of the stream system in tabular format for a user specified reporting interval. This interval will most likely be much greater than the computation interval and as such represents average conditions for the reporting interval. This is a practical limitation with a long-term simulation,based on computer printing cost and the users ability to synthesize the mass of data generated for a given computation interval. The user can specify that the computation interval output be placed on a magnetic storage device for later statistical and graphical analysis by the HEC program WQSTAT.

WQSTAT

Statistical Post-Processor

WQSTAT is a statistical and graphical package consisting of three major components: two graphical plotting programs and a statistical program. Table 2 lists the water quality parameters that can be analyzed by the package. Each of these programs can be executed independently of the other two and each accepts input from a magnetic storage device containing output (at the computation interval) from the Stream Quality Module.

The first plotting program plots the results of the stream simulations as a function of time for any number of these parameters at user specified locations throughout the study reach. The user has the option to superimpose observed values on these plots.

Water quality standards can be specified and plotted as well. In the case of temperature and pH, both maximum and minimum standards values can be plotted.

The second plotting program plots longitudinal profiles of water quality parameters at all nodes throughout the study reach. For each of the eleven parameters, plots of the maximum and/or minimum simulated values over the study period at each node can be generated. The user can specify that observed data and water quality standards be plotted as well. The water quality standards can be changed as a function of river mile to reflect different stream segment quality designations. As a separate option, the user can also input values of the water quality parameters that are indicative of a particular critical quality condition. The

GENERAL CAPABILITIES OF THE STATISTICAL AND GRAPHICAL POST-PROCESSOR

HOURLY STATISTICS GENERATED FOR UP TO ELEVEN WATER QUALITY PARAMETERS

- STREAM FLOW
- WATER TEMPERATURE
- DISSOLVED OXYGEN
- AMMONIA NITROGEN
- NITRATE NITROGEN
- PHOSPHATE PHOSPHORUS
- ALKALINITY
- COLIFORMS
- TOTAL DISSOLVED SOLIDS
- pH
- 5-DAY BIOCHEMICAL OXYGEN DEMAND

TABLE 2

mean error and standard deviation associated with these critical values can also be plotted. The input values for this option generally come from the results of the statistical program.

The Statistical Program has the capability of summarizing by node the maximum, minimum, mean, and standard deviation of the simulated values over the study period, or over a selected sub-period for each of the eleven water quality parameters. It can also compare simulated parameter values against selected water quality standards and determine the number and percent of points exceeding the selected standards. Lastly, it can accept observed data for various locations throughout the study reach and then determine the mean and standard deviation of the difference between the simulated and observed values.

Determination of CSO Impact on the Chemung River

The determination of CSO impact on the Chemung River is based on a long term simulation of meterological, and urban hydrological patterns of the study area. Twenty miles of the Chemung River was selected for study, the study area begins at River mile 23.0 (Fitch Bridge) in West Elmira. This location is approximately 5.0 miles upstream of CSO point source discharges. The study area ends at mile 3.0, approximately 2 miles downstream of a USGS flow gage and NYSDEC water quality sampling station.

The study period selected, dates from June 1966 to October 1975 for 10 years of records. The five month modeling period from June to October is considered to represent the critical period in terms of the river water quality. Daily river flow records are available from the USGS stream gage at Chemung (mile 5.0) for the period, in addition to sparse river quality data. Hourly rainfall data for the period was taken from the Binghamton airport. A comparison study of daily rainfall for Elmira and Binghamton showed a favorable super position, from this it was assumed the rainfall intensity/duration was also comparable and the hourly Binghamton data was taken as representative of the Elmira meterological history.

The drainage basin bordered by the study area includes the Chemung River mainstem and several major tributaries. These include Newtown Creek and Seeley Creek and Sing Sing Creek, just upstream of the study area boundary. Daily flow for the tributaries was developed from USGS recorded flow for Newtown Creek by simple proportion based on drainage area.

The meteorological/hydrological data base consists of 10 years of daily stream flows and hourly rainfall for the months of June through October.

Application of "Storm"

"Storm" determines surface run-off rate and pollutant concentration for rainfall events based on a lumped run-off coefficient aggregated by land usage, and a pollutant accumulation function. The run-off quantity calibration must be accomplished before proceeding with run-off quality calibration. Calibration of Storm for run-off volume is accomplished by adjusting certain coefficient which regulate the volume and timing of run-off. In the coefficient method, the first factor to adjust is the run-off coefficient for the previous areas of the watershed so that the observed and computed magnitudes of run-off volume show fair agreement. Since the coefficient method is a very crude model of the urban run-off process, it may not be meaningful to compare individual event volumes. Other variables that can be adjusted, although to a lesser extent, are the run-off coefficient for the impervious areas and the initial abstraction in the form of depression storage and infiltration.

Calibration of run-off quality can be approached in serveral different ways. One method consists of a sampling program of dust and dirt accumulation and associated pollutant concentrations. Another method involves the determination of daily pollutant accumulation rates and adjustment of wash-off exponents. These two methods apply principally to pollutants associated with surface wash-off, but do not consider the accumulation of material in combined storm and sanitary sewers where the type of deposited material differs significantly from surface pollutants.

The calibration technique employed for this study consists of water quality sampling at CSO point discharge during rainfall run-off events. From sampled data the dust and dirt accumulation rates and wash-off exponent are somewhat arbitrarily set to force an hourly correlation of pollutograph shape and magnitude for several sampled storms. By calibrating on storms with intensity duration relationships comparable to a majority of rainfalls in the data base it can be assumed that Storm will produce reasonable hourly pollutographs for use in receiving water quality studies.

A sensitivity analysis can be performed on this calibration to examine variation in total pollutant concentration for the study period. This is done by varying the calibration parameter that produces a change in the magnitude of predicted pollutant concentration, with the superposition shape remaining unchanged. The Storm model is run with the long term data base with variations in the superposition magnitude, and the resulting long term pollutant concentrations statistics are examined to determine if the statistics are sensitive to the variation. If long term statistics are observed to be insensitive to small (±10% to 20%) variations in pollutant concentration magnitude it can be assumed that the initial calibration produces hourly pollutant concentration which on the average are reasonable.

From here the hourly pollutograph output for the 10 year study period was written to magnetic tape storage for later input to the receiving water quality model.

Calibration of storm for the City of Elmira was based on overflow sampling at a point source to which approximately 1/3 of (1216 acres) the total urban areas is tributary.

Application of "WQRRS"

WQRRS consists of a hydraulics module and a stream quality module. First the hydraulics module is calibrated and run, with the results placed on magnetic tape storage for input to the receiving water quality module. Calibration of both modules is based on available data and engineering judgement. It is unlikely that any receiving water system has or will have been studied in the field to the extent that a dynamic hydraulic and water quality model can be rigorously calibrated. Generally speaking the hydraulics module is more easily calibrated as the hydraulic problem is less complicated, field data is more easily obtained, and knowledge of a few basic parameters may be sufficient for calibration. The stream quality module is many times more complex, inherently involving many more parameters and interrelationships. For this reason, a direct calibration of the stream quality module is not a practical pursuit when dealing with a study level project. If a CSO impact is demonstrated a CSO abatement study should be undertaken at which time in stream sampling during overflow events should be attempted.

Hydraulics Module

The Chemung River was broken into 26 elements varying in length from 0.5 miles to 2.5 miles, the shorter elements being placed in the area of the river believed to be most sensitive to CSO impact. A mainstream base flow daily hydrograph and three tributary base flow daily hydrographs characterize the hydrology of the study area. Two hourly hydrograph/pollutograph inputs are received from STORM. The hydraulics of the study area are developed in the model by the stage flow method, from input characterizing the river channel cross section, slope and friction coefficients. The physical data describing the low flow channel was seen to be lacking in required precision. Therefore, steps were taken to obtain a better hydraulic characterization of the low flow channel. Prior low flow travel time studies were reviewed with NYSDEC to determine if they could be used to benchmark low flow hydraulics. With NYSDEC concurrence on the usefulness of the travel time studies the hydraulic module low flow output was adjusted by formulating a correlation function and using a simple driver computing routine to update the hydraulic characteristics as they are read into the stream quality module.

Stream Quality Module

For a first cut at determining the Elmira CSO impact the stream quality module was set-up for a study level analysis looking at six basic pollutants, these are: suspended solids, settleable solids, BOD, nitrogen, phosphorous, and coliform bacteria. The primary water quality indicators being examined are BOD, dissolved oxygen, temperature, and coliform bacteria. Background levels of these water quality consituents were determined from intermittent sampling data collected at Fitch Bridge (mile 23.0) in West Elmira and Chemung (mile 5.0)

The predictions of changes in water quality constituents as the pollutants are routed downstream are largely uncalibrated. This is the result of insufficient data to calibrate the complex processes occurring in the stream. Therefore, the model predictions are based partly on theoretical knowledge and empirical relationship. The primary empirical function is that of stream reaeration. This is handled in the model by direct input of reaeration coefficients (in a long term analysis this results in essentially steady state, because little data is available) or by selecting one of six reaeration equations. The strategy developed for dynamically determining reaeration coefficients is one of judgment in the comparison of a test run on each of several reaeration equations, the equation producing the most reasonable coefficients over the 26 elements of the stream is selected for the long term simulation.

The results of the stream quality module is a massive listing of water quality constituents concentrations, hour by hour, for each element of the stream for the entire study period. It is obvious that the output in this form is of little practical value in the evaluation of the CSO impact. Therefore, the WQSTAT statistical program package is employed to reduce the output to more usable form. Statistical comparison can be made of modeled results to established stream standards or to the degree and duration of violation of the stream standards. Therein will be found the significance of the CSO impact on the receiving water quality.

TOWARDS STANDARDS FOR COMPUTER-BASED MUNICIPAL DRAINAGE STUDIES

by

William James[1] and Mark A. Robinson[2]

ABSTRACT

The main purposes of a model study are a reasonable understanding of the physical processes involved in the study problem, and a careful evaluation of reasonable alternative designs. The central concern is credibility.

Increasingly, government and municipal engineers control studies in which sophisticated computer models are being used by specialists. Because of the rapid evolution of some of these models, their complex structures and the very large variety of programs available, the situation is becoming increasingly complex. Guidelines for control of the studies may be useful, especially where the local municipal engineer has limited experience with this type of computing.

The following activities could be specified in detail in the initial study terms of reference, even before bids are accepted: problem review, study objectives, performance criteria, requisite accuracy, review of available programs, available data and study resources, program selection criteria, model verification, model calibration, model validation, minimum level of discretization, sensitivity analysis, data preparation and output interpretation, documentation of the modified program actually used, and preparation of machine readable input and output files for archiving.

Each of the above activities is discussed in general terms but the examples used are appropriate to stormwater management modelling studies particularly

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some verification and sensitivity analysis for the hydrological portion of the Stormwater Management Model.

The delivery of computer-based engineering design services could be improved by enlightened study terms of reference and attentive control by the responsible municipal engineers. The suggestions in the paper should reduce the chances of using an inappropriate model, inappropriate use of an otherwise suitable model, such as erroneous data preparation or output interpretation, and ignorance of the model even when the results are acceptable. Credibility would thus be improved.

INTRODUCTION

This paper attempts to set out some special requirements of computer-based studies in municipal drainage. These requirements are additional to those relating to the specifics of the drainage study. The purpose of these additional requirements is to establish the credibility of the computer modelling and to ensure that confidence in the results is transmitted from the engineer doing the computing, or the study, through to those engineers and politicians ultimately responsible for the implementation of the drainage improvements. It is suggested that these special computer-modelling requirements could be set out in the initial Study Terms of Reference. These suggested additional points in the Study Terms of Reference are summarized again at the end of this paper.

Programs such as SWMM are capable of describing drainage hydraulics and hydrology to a high level of system detail. Their strength lies in the fact that they take into account the interaction of a reasonable number of processes and are able to report on the status of certain important variables within these processes throughout the simulation. Of course, there would be little point in using SWMM if the system under study was simple enough to be modelled with the use of cheaper calculating machines than the usual main-frames. The main benefit in using such a complex model therefore appears to be that many, if not all, of the relevant processes and their interactions can be sufficiently accurately investigated and ultimately sufficiently well understood by the engineers responsible for the design. Thus such models requiring main-frame support provide a thorough evaluation of reasonable alternative designs for complex drainage systems. A difficulty that arises is that the person responsible for writing the original program is usually not available for interpreting the performance of the program. Also, the backroom engineers responsible for the computer simulation, often on a considerably modified version of the original program, are frequently remote from the interpretation of the design and ultimate presentation of the results to the public. Whereas the original author of the program, and engineer responsible for submitting the computer runs, may be highly confident of the reliability of his particular computer program, the municipal engineers, and the people responsible for budgeting and supervising the ultimate project may not have the same confidence in the computer methods. Indeed, they may be wise to be sceptical until the results are proven to be sound.

In our work we have tried to establish controls to ensure that the best model is used correctly. This should ensure that credibility and confidence in the results is created and transmitted sequentially from the backroom engineers to the project engineer, perhaps in the consultant's office, and thereafter to the municipal engineers responsible for supervising the study, and ultimately through the engineering committees and other political bodies to the beneficieries, namely the public. In other words, it is not sufficient that the computer model represents all relevant physical processes of the study accurately enough, or that the results are correct or sufficiently accurate. It is highly important that the study be carried out in such a way that there is little chance of using a wrong model, or using wrong data, or wrongly interpreting the results, or simply of not understanding the model or the design.

Of course, there are obvious difficulties. For example, which of the large variety of programs available are best suited to the particular study, or in which order certain models should be used, may be uncertain. Similarly, if the program adopted is very complex, it may not be clear which processes may be safely ignored, or that the data set used is the best of many possibilities.

So we argue herein that many computer-based studies can be enhanced if certain precautions are taken at the outset of the study. We suggest that the Study Terms of Reference include among other things a minimum number of standard tests for verification, sensitivity analysis and output interpretation. Evidently such tests will not materially increase the workload for the consultant, nor will it increase the cost of the study.

COMPLEXITY

Several factors have increased the complexity of computer-based drainage studies over the years:

(1) more and more models are becoming available;

(2) the models are including more processes;

(3) the variety of computing hardware is increasing;

(4) the cost of computing is decreasing;

(5) computer communication between design offices and remote main-frames is becoming easier;

(6) the software capabilities of the computers is becoming more complex; and

(7) there is a professional drive within most engineers to improve their understanding of computer modelling and computer methodology, despite the increasingly longer learning times associated with this methodology, and higher salaries and related costs.

These developments increase the pressure on municipal engineers to use sophisticated design technology which they may not have had time to otherwise use themselves.

Computer Models

Most tried-and-tested programs, such as ILLUDAS, SWMM and HSP have been considerably enhanced recently. For example, the latest versions of each may operate in continuous mode, including water quality processes. Since these programs have been under development for ten years or more, it becomes very important that the precise stage of the evolution of the program be correctly identified when the program is used. Program documentation seldom matches the capabilities of the program at any point in time; consequently it becomes important that the actual program capabilities are accurately and carefully identified.

A recent study[1] lists several hundred computer programs currently available for solving problems associated with water resources development. In fact, it seems that every university and consulting engineering office is bent on establishing their own computer programs for the solution of stormwater management problems. Moreover, while most of the popular models are being enhanced to better describe constituent processes, or to include more processes, other programs describe some of these constituent processes in greater detail. Hence the problem that now arises is the selection of the correct sequence of models to be used in a study. It may be better to use a sequence of process models than to use one of the system models taking into account similar processes, but inaccurately. Evidently few guidelines exist for this model selection process. On the other hand, as a municipality gains experience with one model, there is a strong tendency to prefer studies based on the same model. McPherson describes these problems in a recent paper[2].

Hardware

Readers will be aware of recent advances in programmable calculators[3] and the rapid evolution of micro-computers with their bewildering range of configurations and capabilities. Similar situations exist within the range of mini-computers and main-frames and the machines that lie between these three classes. Increasingly, it is becoming necessary to distribute the computational effort involved in any one study between local micro- or mini-computers and remote main-frames. Many programs are being modified so that their solution methods are better suited to the large, cheap memories and small bit words available on the small systems[4,5]. Again there are few guidelines for this distribution of computational effort.

Systems Ware

New systems software products include local disc operating systems and sophisticated word processors. These products could considerably aid both output interpretation and input data editing. These in turn have benefitted from enhanced communication speeds. In addition, there is an increase in the number of national and international computer networks available, so that the smaller design offices may easily access the largest computers. These trends help smaller consulting engineering offices to gain remote job entry into the large programs maintained by the vendors. With the skills of a recent graduate or postgraduate engineer and an inexpensive terminal, any office has immediate access to the largest computers and the most sophisticated programs on the continent without the burden of maintaining or updating the packages.

STUDY RESOURCES

Individual studies often differ greatly in their objectives and available resources, and consultants usually review the complete study problem and resources at the outset in order to establish the mutually agreed scope and basis for the model study. The problem review should not be done in a general way, capable of ambiguous interpretation. It should be directed so as to list all the hydraulic and hydrological characteristics of the study problem so as to aid in the selection of a computer program. It should attempt to list specific hydraulic and hydrolocial processes that ought to be included in any model study undertaken. For example, if hydraulic jumps or energy dissipation at manholes are important, these should be identified. Few, if any, programs adequately describe these phenomena.

The study objectives and related objective functions should be listed. The purpose of this is to demonstrate how the numbers generated by the computer model relate to the general objectives of the study. Computer models often produce only hydrographs and pollutographs resulting from a certain hydrologic time-series. On the other hand, the study objectives often require a least cost alternative to a specific flooding or drainage problem, such as the production of toxic or other harmful pollutants as a result of rainfall or snowmelt. The review of the objective functions is a clear and logical explanation of how the model study results relate directly to the objectives of the design.

To many observers, the model produces only a sequence of numbers. It is advisable to describe early in the study how these numbers may be compared between several computer runs so as to establish the validity of the computer program. In other words, the performance criteria for assessing the various design proposals must be carefully explained. In some cases it may be sufficient to estimate the total annual loadings whereas in another case the peak flow is more important. The criteria for the performance of one design against another may therefore appear to vary, unless it has been settled initially.

Accuracy is of overriding importance. If the required accuracy can be established early in the study, it would make the selection of the models and the level of discretization for the model study an easier task. In a sense the accuracy level predetermines the model to be used. There seems to be a mismatch between the accuracy of the results demanded from the computer and those obtainable from the field observation program. There is little point in producing simulation results that are much more accurate than the field observations which are used to validate the model. Consider for example the accuracy of rain gauge sampling of storms, and the whole question of design storms. In any case, high accuracy may not be necessary in view of the often tenuous relationship between the study objectives and the performance criteria.

Review of Models

There is a large amount of published literature and data on the performance of various models[1,2,6]. A simple review of the documentation will indicate the accuracy with which selected processes are modelled. The engineers conducting the study should attempt to show why selected models are deficient or sufficiently accurate for their purposes in the light of the problem review and special hydrologic and hydraulic processes that are required for their particular study. The purpose here is to show how models may be used to support one another, or to establish a reasonable sequence of model use in the study. No model selection should be undertaken previous to this stage; first of all review all available data and available study resources including the models, then establish the criteria for selecting the models and finally select the model sequence.

It is much simpler to seek out and review the available data than to collect data from scratch. In two recent Canadian studies several years of sewer water level records as well as rainfall records were surprising finds. Such data could form part of the model selection criteria; to make maximum use of available data the study should not be constrained to use a model unsuited to these (unknown) data.

The initial review should include a description of available manpower, time, and money resources; the management of a study is an attempt to maximize the level of detail subject to manpower, time and money constraints.

Model selection criteria should be clearly stated in the study report. It is easy to criticize a study at a later date for not using a model which would describe processes to a much higher level of system detail if it is not made clear that the simpler model was used because of limited time and money. Other criteria include availability of the model, availability of advice, experience with similar or alternative models, availability of data, and costs.

MODEL CREDIBILITY

An additional series of verification, validation and sensitivity tests will produce sufficient information to answer a wide range of questions about the model's performance.

Verification Tests

Verification tests use some specific conditions for which the model response can be exactly predicted to check if indeed the model is structured as intended. Verification tests are not conducted by comparison of model responses with those of the actual system to be modelled; rather, comparisons between model responses and theoretically anticipated results are made in as many cases as possible. The input data need not be physically reasonable.

It would be useful to create a standard input file for verification tests for stormwater models. A hypothetical system comprising two simple, square sub-catchments, of say one acre each would be suitable. The sub-catchments could be joined by a pipe of standard diameter and simple form. The hydrograph from the first sub-catchment is attenuated in the pipe. At the downstream end of this pipe the hydrographs from both sub-catchments are superimposed. At the outlet of the pipe, the combined hydrograph could be routed through a simple standard storage tank. The purpose of this verification data set would be simply to test the algorithms for:

(1) the generation of the overland flow hydrograph;

(2) the addition of two hydrographs;

(3) routing in the pipes; and

(4) storage routing in storage tanks.

For example, we have used the following tests:

(1) zero rain to ensure that no runoff is generated;

(2) very steep catchments to ensure that the hydrographs generated are very similar to the input hyetograph;

(3) light rain and high infiltration rates to ensure that no runoff hydrographs are generated;

(4) completely impervious catchments to ensure that the total volume of runoff is equal to the total volume of rainfall;

(5) low infiltration rates and high runoff to ensure that the correct amount of infiltration is subtracted;

(6) very flat pipe gradients to check the surcharge calculation;

(7) similar tests with small diameter pipes and high pipe roughness;

(8) high and low initial abstractions; and

(9) tests on the storage routing parameters, viz. the outlet rating curve and the storage curve.

In all of these tests particular attention is paid to the summary output: total precipitation, total gutter flow, total snowmelt, total infiltration, etc. This is an essential check that the numbers generated by the computer are sensible.

Perhaps special algorithms and data files should be built into the model. When the appropriate button is pushed the model automatically carries out a series of verification tests. In our undergraduate classes the following verification tests are normally carried out prior to any analysis being done:

1	Low imperviousness	5%
2	High imperviousness	95%
3	High detention	0.62 - 1.84 (in)
4	Low detention	0.01 - 0.1 (in)
5	PCTZER (High)	95%
6	PCTZER (Low)	5%
7	High continuous rain	10 inch/hour
8	Low continuous rain	0.1 inch/hour
9	High continuous infiltration	2 inch/hour
10	Low continuous infiltration	0.1inch/hour

Validation and Calibration Tests

Validation implies the comparison of results to field measurements, to another model known to be accurate, or to some other adequate criteria to ensure that the model is producing accurate data. If these comparisons indicate that the model results are not sufficiently accurate, the model is altered and the procedure is repeated. This validation process generally involves several iterations before a satisfactory confidence level is achieved. A recent paper describes the process for combined quantity and quality modelling[7]. Techniques used in validation include:

(1) validation of parameters and results against field observations;

(2) cross correlation of model results with those of another proved, usually discrete event model; and

(3) some combination of field observations and field modelling.

The most accurate method of validation is the comparison of output from the verified model against corresponding field measurements.

The validation process should be limited to the range of parameter values applicable to the normal operating conditions of the system. First, acceptable tolerances must be established. They should be related to achieveable field observations and the accuracy of field equipment. Then, emphasis should be placed upon critical parameters -- those that have the greatest effect on the performance of the system. Reasonable assumptions may be satisfactory for less critical parameters.

The procedure, which may be necessary, of improving empirical constants in the model to achieve proper correlations is also called calibration. Once verified and calibrated, the model can be applied with confidence in the evaluation of the real system. Important parameters are:

(1) percentage imperviousness
(2) width of sub-catchments
(3) initial infiltration rate
(4) final infiltration rate
(5) infiltration decay rate
(6) ground slope
(7) Manning's roughness for overland flow
(8) depression storage for both pervious and impervious areas.

Validation tests using a full input data set may provide a useful test of the logic underlying the data set.

Level of Discretization

The procedure for systematic disaggregation has been described in an earlier paper by the author[8]. Disaggregation implies more sub-catchments of smaller area and a finer time step. For coupling the time increment to the size of the sub-catchment elements, the concept of an impulse response function is useful, i.e. an instantaneous unit hydrograph. For example, it may be accurate enough for our purposes to represent this response function by a time vector of 20 elements. Sensitivity tests in which the time step is systematically changed may also be appropriate. In practice, disaggregation and sensitivity analysis proceed simultaneously[8]. Often, careful disaggregation and appropriate selection of the time step will produce better results than extensive optimization of empirical factors such as Manning's roughness for pipes and overland flow. A recent paper[9] by Alley and Veenhuis provides some guidelines.

There is an obvious conflict in the selection of the level of discretization. Consultants' costs increase rather rapidly with an increasing number of sub-catchments. More data has to be abstracted and prepared, and more expensive computer runs will result. On the other hand, the client will usually prefer to have a high level of discretization for the agreed fixed study price. It is useful to have, at the outset of a study, an indication of the desirable level. As a guide, it is important to identify all hydrologically significant elements in the system, where it is necessary for hydrographs to be generated. If these elements can be identified at the time the terms of reference are set out, consultants will have a better idea of the scope of the work. It makes sense to face this possible conflict squarely at the beginning of the study.

Sensitivity Analysis

Sensitivity analysis proceeds by holding all parameters but one constant at their expected values, and perturbing that parameter within reasonable expected limits such that the variation of the objective function can be examined. If what appear to be small perturbations of the parameter produce large changes in the objective function, the system is said to be sensitive to that parameter. The user must obtain a measure of how accurate that parameter must be represented in his model. If the objective function is not sensitive to the pertubated parameter, then the parameter need not be accurately represented. If the system is insensitive to the pertubated parameter, the parameter and its associated process is redundant and the process should be deleted. It must be stressed that the actual values of the constant parameters may affect the sensitivity analysis and so their values should be typical of the conditions being modelled.

Here again algorithms should be available that permit the user to easily conduct a sensitivity analysis. When another button is pushed, the computer requests the user to identify the parameter whose sensitivity is to be tested, and the range of perturbed values. The data file will be automatically rebuilt and the tests carried out. In addition, all output hydrographs should be plotted on the same family of curves in order to present the impact immediately to the user.

Essentially there are two components of the hydrograph: peak flow and runoff volume. The parameters which affect each component are as follows:

A: Peak Flow: slopes (pipe and land)
Manning's n
overland flow width
pipe length

B: Volume: infiltration
surface retention
PCTZER
% impervious

Control of Errors

Of course, it is not sufficient merely to carry out the required verification, validation, and sensitivity analyses to various levels of discretization. These results have to be presented in the report in a way which ensures that the purpose of the tests has been achieved. The author of the report must explain the results, in other words, interpret the results in a satisfactory way. The verification tests must be shown to produce expected results. The calibration and validation results must be shown to be reasonable. The trends resulting from the sensitivity analysis must be shown to make sense. The purpose of this is to provide evidence that the model is indeed performing in a reasonable way: (a) the verification tests demonstrate that there are no serious errors in the coding of the model, (b) the validation tests at a simple level of discretization, say one or two sub-catchments and pipes, demonstrate that the model is being used in a in a reasonable way, (c) the validation tests on the full data set, i.e. for all of the sub-catchments acting together as a hydrological system, indicate whether serious blunders have been made in preparing the input data set, and (d) the sensitivity analyses will indicate whether the level of effort put into estimating the individual parameters is appropriate based on their significance in affecting model results.

The required output interpretation, arguing that the results in fact make sense, helps to ensure that the authors have not misinterpreted the model results.

CONCLUSIONS

In this paper we have suggested that the following topics should be included in an initial terms of reference:

Problem Review - The problem review should identify all hydraulically and hydrologically significant elements in the study area so that the model selected can be shown to include all relevant processes.

Study Objectives - The study objectives should be reviewed to show how the objective functions, viz. pollutographs and hydrographs, relate to the design alternatives.

Performance Criteria - The performance criteria for the comparison of one design alternative against another must be correctly identified so that the simplest possible model can be justifiably selected.

Requisite Accuracy - Accuracy of field measurements for validation should be carefully reviewed in order to ensure that the model runs are not inordinately expensive.

Review of Available Programs - Several programs should be suggested or selected for review. The review should consider process models as well as system models, and a sequence of models.

Study Resources - Study resources include time, manpower and money and these, in turn, will determine which of the models may be selected.

Model Verification - Verification tests should be required on a simple data set consisting of two small catchments connected by a simple pipe and feeding into a simple storage tank. The verification tests should demonstrate that the coding is performing as intended.

Model Calibration and Validation - Validation tests should be carried out on one of the sub-catchment data sets to demonstrate that the model is being correctly used. Validation tests should be carried out on the full data set to demonstrate that the input data are reasonable.

Minimum Level of Discretization - The smallest number of sub-catchments required for modelling the system should be selected commensurate with the objectives of achieving the best design at a reasonable cost. These minimum levels should correspond to the disaggregation necessary to identify hydrographs at all hydrologically significant elements in the drainage system, which could be listed.

Sensitivity Analysis - Sensitivity analyses should be carried out on a minimum number of parameters, for example, infiltration parameters, roughness values, widths of sub-catchments, etc., to identify which are of most significance and hence to justify the effort put into their estimation.

Data Preparation and Output Interpretation - All output should be interpreted to demonstrate that the model is performing in a logical way.

Documentation - The version of the program actually used in the study should be identified and appropriate documentation sources listed in the report. In addition, the machine readable input and output files should be archived for future use.

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SIMULATION OF EFFECTS OF URBANIZATION ON STORMWATER HYDROGRAPHS AND POLLUTOGRAPHS-- A REGIONALIZED PARAMETRIC APPROACH

by

Donald E. Overton[1], William L. Troxler[2] and Ernest C. Crosby[3]

The University of Tennessee Runoff Model (TENN-I) for simulating storm hydrographs and pollutant yields was developed in the course of performing three separate but complimentary studies. All three studies had the same fundamental objective, i.e. to evaluate the effects of specialized land use on stormwater runoff and its associated quality.

The three studies were funded by the following three Federal Agencies and dealt with the land use indicated:

	Federal Agency	Contract-Grant No.	Study Period	Land Use
(1)	U.S. Department of Energy	EY-76S-05-4946	1975-79	Coal Strip Mining
(2)	U.S. Department of Interior Office of Water Resources Technology	TENN-A-046	1976-78	Urbanization
(3)	U.S. Air Force	AF FO 8635C 77	1975-79	Air Force Bases

TENN-I was developed in the course of analyzing 410 storms observed on 36 watersheds. The storm sample included watersheds in agricultural, urban and 100% forested land use conditions as well as watersheds undergoing coal strip mining and watersheds at a U.S. Air Force Base.

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2. Objective of TENN-I

TENN-I was developed for the purpose of simulating stormwater hydrographs from a real time or design storm rainfall time distribution and land use and soil type of the watershed of interest. The watershed is considered to be a lumped system, and the required basin characteristics are: percent of watershed that is forested (PF), percent that is impervious (PI), percent in strip mining or denuded (PS), and surface drainage area in square miles (SQMI).

Runoff volume and the associated rainfall excess time distribution are simulated from the input rainfall using the U.S. Soil Conservation Service Curve Number model (CN) [1]. If a runoff hydrograph is read into the program associated with a rainfall hyetograph, the program will compute a CN from this information. Otherwise, a CN must be read into the computer, and would be simulated using the procedures specified in reference [1].

TENN-I makes provision for simulating a unit hydrograph or unit response function, URF, which is convoluted with the rainfall excess hyetograph simulated using the CN model. This computer program is an adaptation of the simulation phase of the TVA double triangle model reported by Ardis [2] and later modified by Betson [3]. A more detailed description of the developement of TENN-I has been reported by Overton, Troxler and Crosby [4] and Overton and Crosby [5].

TENN-I also simulates pollutant loads for the associated storm. This simulation is a function of the above specified watershed and storm characteristics. Hence, additional parameters need not to be read in.

3. Simulation of Storm Hydrograph

a. Normalized Unit Response Function (NURF)

The URF in TENN-I is based upon Ardis' [2] quadrilateral function. The URF was coupled with the CN model to form the TVA double triangle model. The shape of the URFs and associated CN have been optimized on a total of 410 storms in studies performed by Ardis [2], Betson [3], Overton, Troxler and Crosby [4] and Overton and Crosby [5]. Optimizations were performed by a pattern search routine.

The quadrilateral URF was based on the concept of partial-area runoff which assumes that the initial or quick response from a watershed comes from the riparian areas. As other areas of the watershed become saturated, they too begin to contribute to runoff in the form of a delayed response.

Ardis [2] assumed that these two responses could be simulated by two separate triangle response functions as shown in Figure 1. When added together, these two triangles form a quadrilateral unit response function for the storm as shown in Figure 2.

In deriving the URF, it was assumed that (1) the peak of the delayed response (UR) occurs at the end of the initial response (T2), and (2) the time bases of both responses and the time to peak of the initial response (T1) must be integer multiples of DT. No assumption was made concerning the relative volumes contained in the initial and delayed responses or concerning the relative magnitudes of the peaks of the invidivual responses.

The double triangle URF is defined by the five parameters UP, UR, T1, T2, and T3. T3 is determined by:

$$T3 = (NOBS - NRAIN + 1) * DT \qquad (1)$$

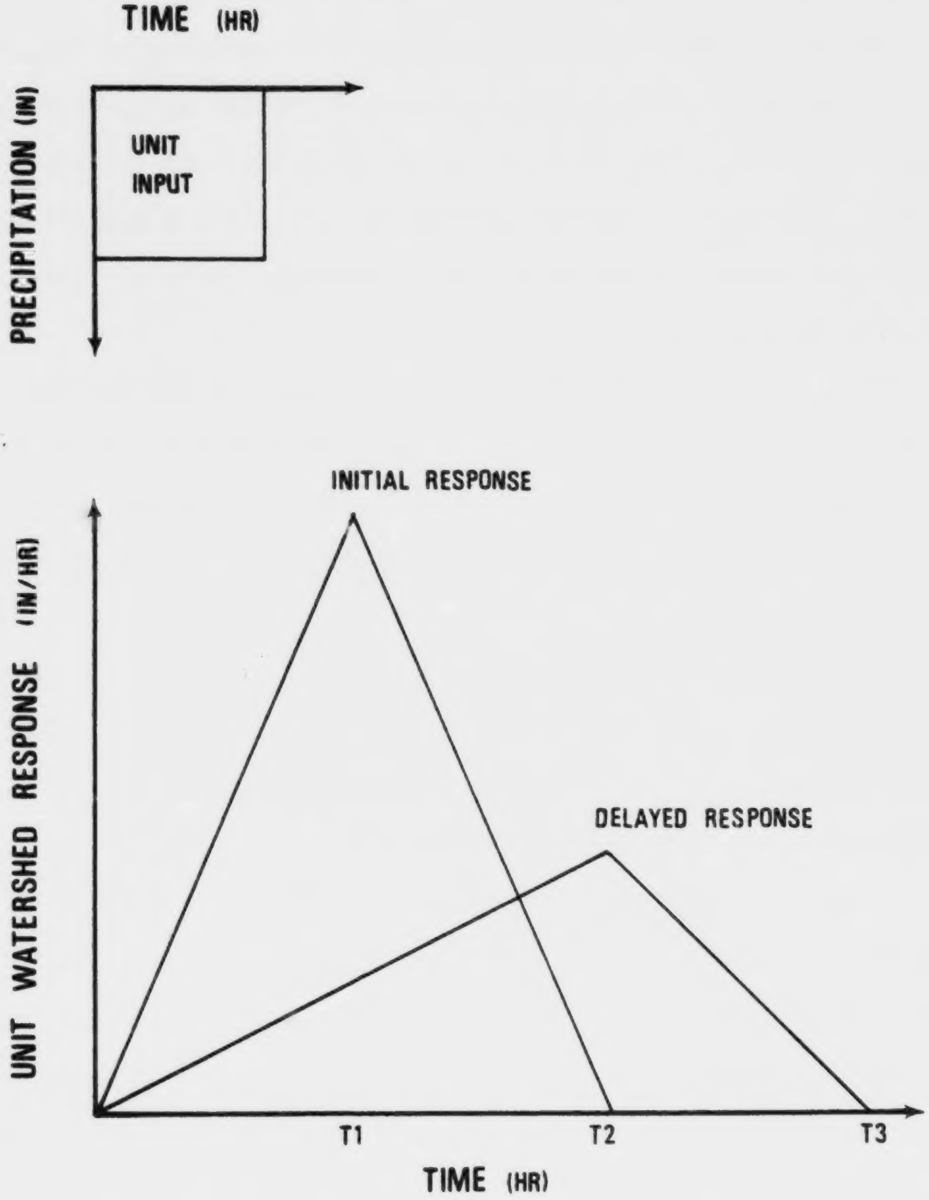
TIME (HR)
PRECIPITATION (IN)
UNIT
INPUT
INITIAL RESPONSE
DELAYED RESPONSE
UNIT WATERSHED RESPONSE (IN/HR)
T1
T2
T3
TIME (HR)

Figure 1. Partial Area Runoff Concept Represented by an Initial and Delayed Response (After Ardis (2))

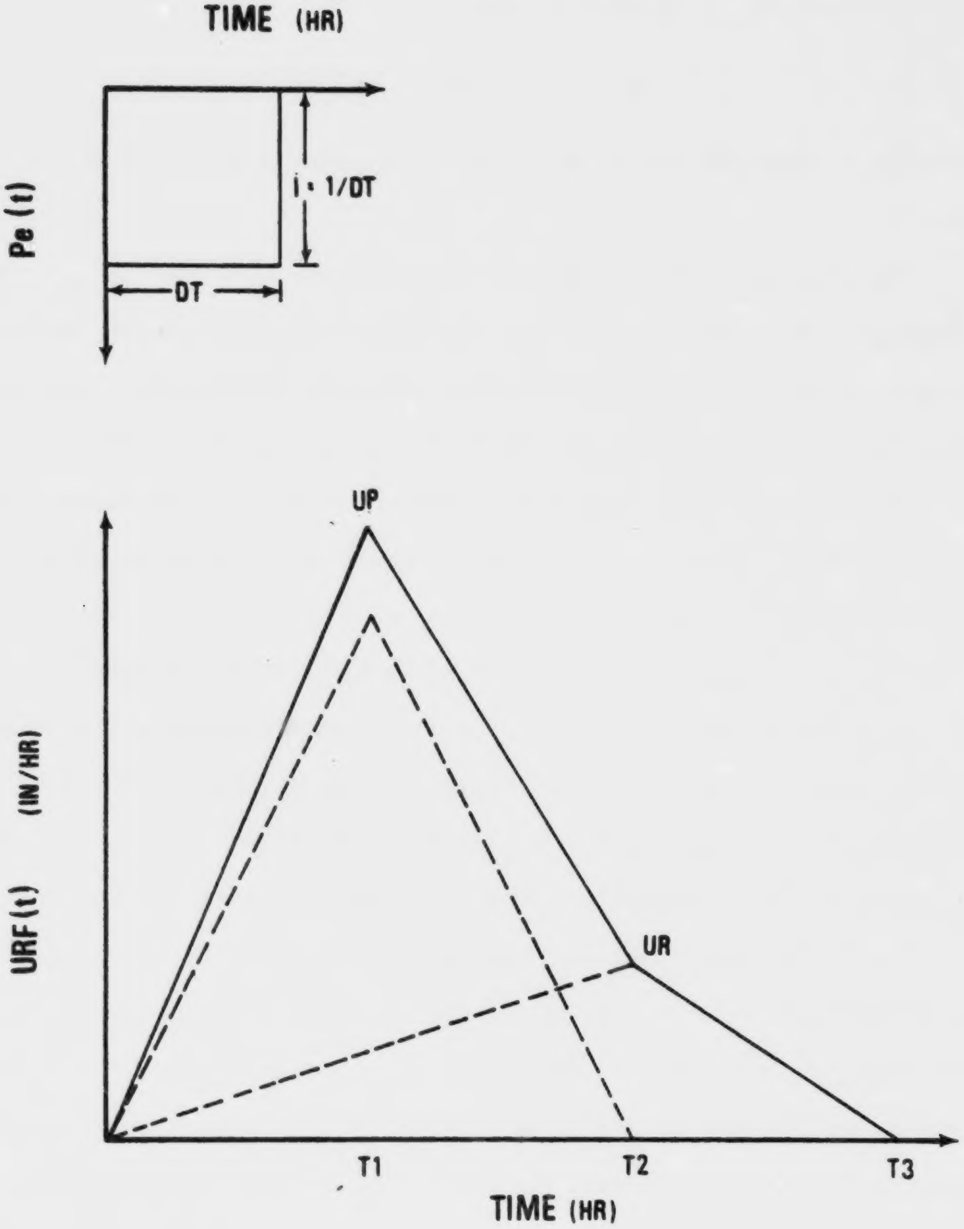
TIME (HR)
Pe (t)
i = 1/DT
DT
UP
UR
URF(t) (IN/HR)
T1
T2
T3
TIME (HR)

Figure 2. Double Triangle Model for Unit Response Function
(After Ardis (2))

where NOBS = number of storm hydrograph ordinates in multiples of DT and NRAIN = number of rainfall increments in multiples of DT. By maintaining a unit volume, UR is calculated from:

$$UR = (2 - (UP*T2))/(T3-T1) \qquad (2)$$

Defining a storm URF therefore involves determining values of UP, T1, and T2.

The parameters UP, T1, and T2 were optimized using the pattern search technique. The objective function was the minimization of the sum-of squares of errors between observed and simulated discharges. Since all five parameters describing the model were allowed to vary from storm to storm, the model is, considered nonlinear. Rainfall excess was optimized using the SCS-Curve Number model after setting it equal to the observed direct runoff volume.

The variability of the URF from storm to storm within a watershed was expalined by normalizing the time and discharge scale by the associated URF lag time, TL, where TL = time lapse between occurrance of 50% of the rainfall excess block and 50% (or 1/2 inch) of the URF volume, as shown in Figure 3. These normalized URFs are hereafter referred to as NURFs.

A NURF for each major land use category was identified and they are shown in Figure 4. The categories are, (1) strip mined, (2) 100% forest, (3) urban without extensive storm sewers, (4) urban with extensive storm sewers, and (5) agricultural. As a matter of providing a reference or bench mark, the NURF observed for sheet surface runoff from a plane, reported by Overton and Meadows [6], and the NURF derived for the V-shaped watershed of Overton and Brakensiek [7] (see Figure 5) from the kinematic wave equations are also shown. The stripmined watersheds were in

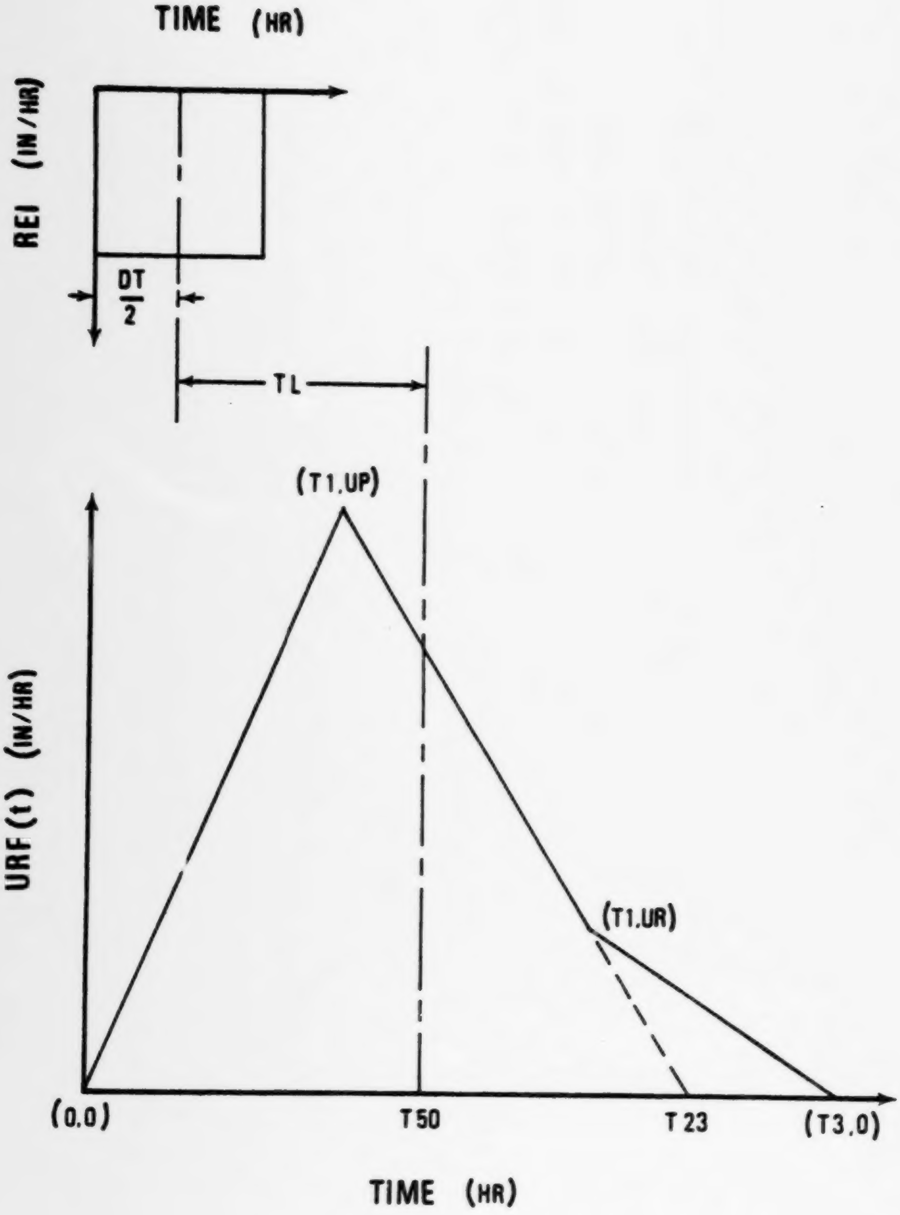
TIME (HR)
REI (IN/HR)
DT/2
TL
(T1,UP)
URF(t) (IN/HR)
(T1,UR)
(0,0)
T50
T23
(T3,0)
TIME (HR)

Figure 3. Evaluating Lag Time of the Double Triangle Unit Response Function

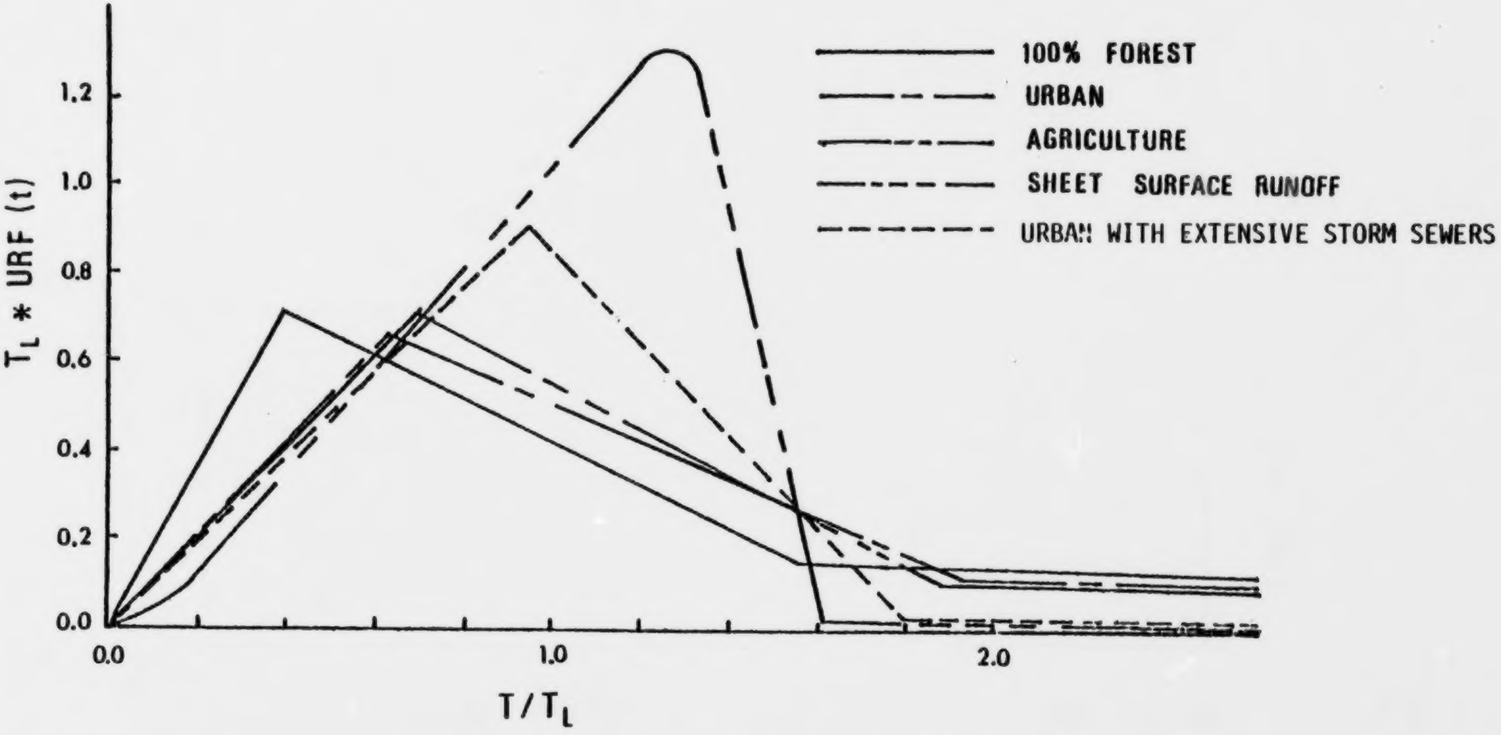

Figure 4. Normalized Unit Response Functions for Different Land Uses.

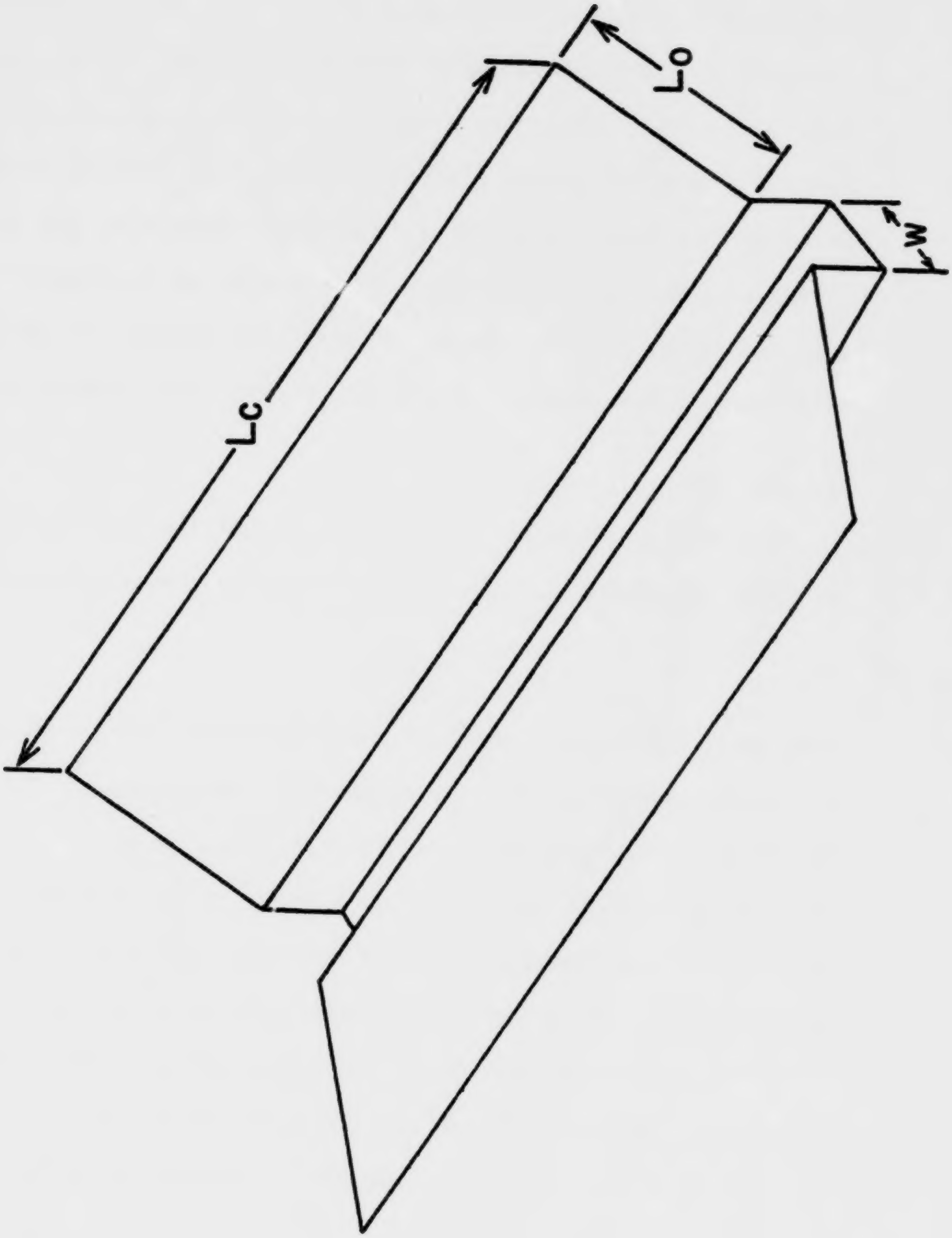

Figure 5. A V-shaped Watershed (After Overton and Brakensiek (1973))

previously 100% forested areas. Hence, a pattern is shown where the forested and strip mined watersheds have a small initial response whereas the urban and agricultural watersheds have a larger initial response. This implies that imperviousness and crop land produce much more surface runoff. The storm sewered NURF has an even higher initial response (80%) which is more likely generated by the runoff collection and rapid transport to the watershed outlet. Sheet surface runoff has no delayed response as does the V-shaped NURF. Hence, the variation between the two NURFs is attributed to the geometry of the flow path of the V-shaped watershed.

b. Lag Time

Lag time for a storm is simulated in TENN-I using the concept that it varies inversely with the generating rainfall excess intensity.

$$TL \sim i_e^{-b} \qquad (3)$$

This variation is well known for sheet surface runoff of overland flow [6] and documentation of this variation for watersheds is growing [5,6]. The first such documentation of lag time variation on a watershed was reported by Minshall [8] and is represented by the five URFs he obtained from each of the five storms (see Figure 6). Lag time is increasing with decreasing rainfall excess rate. These URFs were derived from storms of 10 minute duration as opposed to the long duration storms from which the URFs were optimized in the noted studies [2, 3, 4, 5].

For sheet surface runoff, lag time is related to input intensity as

$$TL\ (\text{min}) = 0.58\ (nL/\sqrt{S_0})^{0.6}/i_e^{0.4} \qquad (4)$$

Eq. (4) can be derived from the kinematic wave equations [6], and lag modulus, μ, is defined as

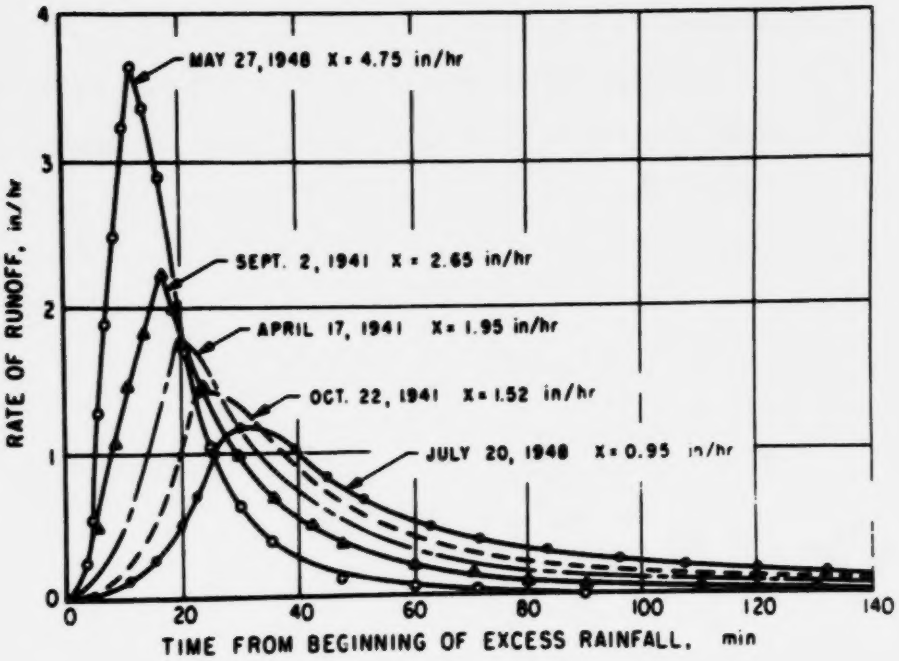

FIGURE 6: Variation in Unit Hydrograph as a Function of Rainfall Excess Intensity (After Minshall [8])

$$\mu = 0.58 \left[\frac{nL}{\sqrt{S_o}}\right]^{0.6} \tag{5}$$

where n = Manning roughness coefficient, L = length of plane in feet and S_o = slope of plane.

Lag modulus is solely a function of the geometry and hydraulic roughness of the plane.

The lag modulus model is used in TENN-I as an analogy for watershed runoff and standardized to turbulent sheet surface runoff by fixing the exponent on input intensity at 0.4. Lag modulus was optimized for all 36 watersheds [4, 5] using a weighted storm rainfall intensity, WRE,

$$\text{WRE} = \sum_{j=1}^{N} i_e^{2}(j) / \sum_{j=1}^{N} i_e(j) \tag{6}$$

where N is the number of time intervals equal to DT

For watersheds, Eq. (1) becomes

$$\hat{\text{TL}}\ (\text{min}) = \mu/\text{WRE}^{0.4} \tag{7}$$

Lag modulus was related to watershed characteristics in the following manner:

Rural Watersheds

$$\hat{\mu}\ (\text{hrs}) = 0.060 * \text{SQMI} + 0.0203 * \text{PF} + 1.16 \tag{8}$$

where PF = % forest

Urban Watersheds

$$\hat{\mu}\ (\text{hrs}) = 3.24\ [\text{SQMI}/\text{PI}]^{0.6} \tag{9}$$

where impervious

c. Convolution of the URF with Rainfall Excess Time Distribution

The final step in simulating a stormwater hydrograph is to convolute the storm URF with the rainfall excess time distribution. The URF is defined by simulating storm lag time from Eq. (7) using either Eqs. (8) or (9) depending on the watershed land use. WRE is calculated by Eq. (6).

4. Simulation of Pollutant Yield

a. Load Modulus

TENN-I simulates storm pollutant yield using a load modulus (lbs/acre-in of storm runoff) as a function of percent stripped or denuded, PS, lag modulus and percent forest or trees in the following form:

$$\rho_W = C_1 * PS - C_2 * \mu * PF + C_3 \qquad (10)$$

The coefficients were optimized using stormwater quality data on a total of eleven watersheds, six undergoing coal strip mining and five urbanized [5].

Eq. (10) was derived from a mass balance, and each of its terms represents a component of pollutant yield.

$C1 * PS$ = source of pollutant or soil loss

$C_2 * \mu * PF$ = deposition between source and outfall, and

C_3 = storage in watershed picked up and redeposited.

Eq. (10) is implicitly based upon a plug flow or first in first out inventory concept where the processes are in equilibrium.

The coefficients optimized for the coal strip mined and urbanized watersheds are shown in Table 1.

b. Storm Load

Once load modulus for the watershed pollutant has been simulated, the storm pollutant yield, SPY, is simulated by

$$SPY = \mu_w * AREA * 640 * SRO \qquad (11)$$

where SRO is the total storm runoff in surface inches, and AREA is in acres.

Table 1

Coefficients in Pollutant Yield Model (Equation 10)

URBAN [Ref. 4]

	Coefficients		
Pollutant	Source C_1	Deposition C_2	Storage C_3
Suspended Solids	16.7	21.5	62.3
FE	0.442	0.568	1.54
MN	0.0072	0.0092	0.20
CA	0.147	0.189	2.45
MG	0.0597	0.113	0.0323
Sulfate	0.0719	0.0216	1.24
Total Alkalinity	0.319	0.128	6.71

COAL STRIP MINED (Active) [Ref. 5]

Suspended Solids	18.2	1.28	576.7
FE	0.323	0	0.12
MN	0.0161	0	0.002
CA	0.131	0	0.38
MG	0.133	0	0.40
Sulfate	1.39	0	2.45
Total Alkalinity	0.19	0	2.00

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STORMWATER MODELLING APPLICATIONS
IN THE CITY OF EDMONTON

BY

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U.S.A.

STORMWATER MODELLING APPLICATIONS IN THE CITY OF EDMONTON

by

M. AHMAD[1]

INTRODUCTION

The purpose of this paper is to present an overview of the various modelling applications recently carried out by Edmonton Water and Sanitation (Drainage Engineering Section).

The City of Edmonton has played a leading role in the implementation of new modelling technology, and is among the first cities in Canada to use sophisticated hydrologic simulation models for planning and design purposes. Edmonton Water and Sanitation has been using the SWMM/WRE [1,2] models since 1975 to upgrade the level of flooding protection in the existing combined sewer areas.

Initially, the use of computer models was restricted to relief sewer studies. At present, however, the City is employing a set of models including SWMM/WRE [1,2], STORM [3], HYMO [4], and ILLUDAS [5] in a variety of applications such as: (1) preparation of master drainage plans for the combined and separated sewer areas, (2) development of a long term program to provide relief to the existing overloaded combined sewer systems, (3) evaluation of combined sewer pollution abatement strategies, (4) development of a stormwater management criteria for new areas, (5) analyzing small watersheds for drainage planning purposes, and (6) verifying new system designs in newly developing areas prior to construction.

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EDMONTON SEWER SYSTEM

The City of Edmonton is the capital of oil rich Canadian Province of Alberta. It has a population of about 500,000 and is presently growing at a rate of 2.0% per year. The City is located along the banks of the North Saskatchewan River which divides the City into two parts - Northside and Southside - as shown in Figure 1. At present, approximately 73% of the total City area (123.0 sq. mi.) is fully developed. The remaining area is developing at an increasing rate of about 1300 acres/year. Over 2,200 miles of sewers (including sanitary, storm and combined) service the developed areas of the City. About 20% of the total developed area in the City is drained by the combined sewers while the remainder by the separated storm sewer systems. Figure 1 also shows the location of the combined sewer area in the City. Most of the presently undeveloped areas located within the City Limits are drained either by the existing storm sewers or by the tributaries of the North Saskatchewan River such as Mill Creek, Fulton Creek, Whitemud Creek, and Blackmud Creek.

The existing combined trunk sewer system drains an area of approximately 12,000 acres which is located in the older central core of the City, which was developed prior to the early 1950's. During storm events this system overflows to the river at 12 locations and at one location to Mill Creek. The combined sewer system is characterized by deep tunnels (up to 125 feet below ground) ranging in size from 48" to 126" and connected to local lateral and collector sewers through drop structures. The existing combined trunk sewer system is very complex and consists of a large number of overflow weirs and interconnecting conduits. Portions of this system are over 60 years old. Continual development and redevelopment in the combined sewer areas gradually increased the imperviousness ratio resulting in the creation of runoff greatly beyond the design considerations of the original sewer system. These increases in runoff due to increased impervious areas have caused overloading of the existing combined sewers with the resultant surcharging causing frequent basement and surface flooding problems.

The existing combined sewer system is intercepted by two major interceptors, one serving the combined system on the north of the river, and the other, the system lying south of the river. These interceptors converge upstream of the Gold Bar Waste Treatment Plant.

The Gold Bar treatment plant provides treatment to approximately 92% of the City's sanitary sewage flow plus intercepted storm flow from the combined sewerage system. This plant has a design capacity of 43.5 MIGD. The average daily dry weather flow to plant is approximately 48.8 MIGD. An extensive expansion program is currently underway which will expand the design capacity to approximately 69 MIGD during the coming year which will provide adequate capacity to accomodate the City's needs until approximately 1989.

MODELLING APPLICATIONS

1. Combined Area Relief Studies

Combined Drainage Area Study [6]:
The first modelling study was commissioned in 1975 to investigate storm drainage conditions and to determine relief requirements in the combined sewer districts of the City. Prior to that time it was considered that total separation of the existing combined system would be required to provide effective relief. However, it was recognized that the unusual and complex combined sewer system in the City could not be analyzed with traditional methods. It was, therefore, necessary to use the then new computer modelling methods for the analysis and design of relief sewers in this area. The original version of the WRE model [2] which was the most appropriate model then available was used in this study. This study concluded that neither major trunk sewers nor sewer separation were required for hydraulic relief and the majority of the flooding problems in the combined sewer area were found to be largely due to inadequate local sewers. The study also concluded that considerable in-system storage capacity was available for possible control of combined sewer overflows. However, shortly after the completion of this study, it was discovered that the WRE model overestimated the pipe system storage in surcharged sewers. Subsequently, modelling results were updated using the corrected version of the WRE model in an in-house

study [7] completed last year. Revised modelling results indicated that the original study underestimated the trunk relief requirements and a more extensive relief program than previously determined would be needed. To finalize the combined trunk relief requirements an update study will be conducted this year.

Combined Area Relief Program:
Two earlier studies completed in 1976 [6] and 1979 [7] indicated that the relief requirements for the existing combined sewer area are two-fold: firstly, the major trunks and secondly, the lateral and collector sewers. A 22-year long range program for relief and upgrading of the existing overloaded combined lateral and collector sewers was adopted by City Council and initiated in 1978 after determining the relief requirements for individual subareas within the combined sewer area. The entire 12,000 acre combined area was divided into 32 drainage subareas which were then analyzed and ranked according to the assessed priority for relief. The priority ranking analysis was based on an evaluation of a number of factors for each subarea including sewer density, length of the existing relief sewers, reported basement flooding frequency, potential for redevelopment and need for environmental improvements. The entire combined sewer area has been divided into three groups, namely, high, moderate and low, depending on the need and urgency of the required relief. On the basis of this priority ranking analysis, an annual construction program has been established for each year until year 2000, with specific areas identified for each year's construction. This relief construction schedule has been adopted by City Council, and hopefully will not be significantly altered to satisfy independent political needs in the future.

Relief sewer construction in the combined sewer subareas is tentatively scheduled for completion by the end of year 2000 and the detailed relief analysis and design will be completed by 1986. Relief construction schedule for the combined trunks will be finalized upon completion of the update study which is to be completed by mid 1980. The present relief program is based on a relief sewer construction rate of 500 acres per year. Relief construction has already been completed in more than 1500 acres of the combined area.

Financing for total cost of this relief program is being derived from a special re-development assessment levy charged against all re-developing properties. It is not anticipated that any portion of the cost of this program will be financed through the general tax levy.

The total construction cost for providing relief to the existing combined lateral and collector sewers is estimated at approximately $40 million (1977 dollars). An additional cost of about $1.3 million was estimated in the 1976 study [6] in order to relieve the existing combined trunk sewers. A cost comparison for two relief alternatives namely, complete sewer separation and combined relief is presented in Table 1 [8].

Table 1

COST COMPARISON FOR
RELIEF ALTERNATIVES

	RELIEF ALTERNATIVE	
	SEWER SEPARATION ($)	COMBINED ($)
Laterals & Collectors	104,000,000	40,000,000
Trunks & Interceptors	60,500,000	1,300,000
Overflow Control Structures	-	7,700,000
Total	164,500,000	49,000,000

NOTE: Costs in 1977 dollars

From Table 1 it is evident that alternative 2 (combined) is the most cost-effective solution. However, results of the update study (to be completed in 1980) may indicate that trunk relief costs are considerably higher than those originally estimated. Relief costs for the combined lateral and collector sewers were estimated using a simple relationship between "impervious storage factor (ISF)" and relief cost in dollars per acre for a subarea. A relationship as shown in Figure 2 was developed using the actual cost data for three subareas, Calder, Norwood and Alberta Ave/Eastwood. Relief construction in these areas was completed prior to 1978. Impervious Storage Factor for a drainage area can be calculated using the following equation:

$$\text{I.S.F.} = \frac{\Sigma\, l_i d_i^2}{\text{I.A.}}$$

where

l_i = total length in feet of sewers of diameter d_i

d_i = diameter of a sewer in feet

I = imperviousness ratio of the area

A = drainage area in acres

Impervious Storage Factor, in general, reflects the drainage characteristics of a catchment by relating the pipe-full storage available within the sewer system to the upstream contributing impervious area. This concept was initially used in the City of Winnipeg Relief Study [9].

Estimated relief cost for the recently analyzed Garneau area is found to be in close agreement with that originally estimated using ISF curve. However, actual relief construction cost in the Northcote area in which construction was completed in 1979 was much lower than originally estimated mainly due to overestimation of the imperviousness ratio used in computating ISF value which resulted in higher estimated cost for relief.

2. Combined Sewer Pollution Abatement Study [10]

As a follow-up to the first study in 1976 [6] an extension study was carried out to determine the feasibility of controlling surcharge and storage in the deep tunnel system in order to reduce the number of overflows from the combined sewage overflowing to the North Saskatchewan River and to partly equalize flows to the treatment plant. The number of overflows from the combined system was simulated over 1 year period using the STORM model which was calibrated against overflow measurements made by the City in 1975. The modelling results indicated that relatively small volumes of storage would be quite effective in reducing overflows and overflow pollution. Subsequently, more detailed models were employed to assess the effectiveness of employing a variable height regulator gate to control storage in the sewer system, and for an accurate assessment of first flush pollutant loadings. Automatic regulators in the other cities were investigated and a programme for the implementation of computer controlled variable regulator gates at locations throughout the combined sewer system was developed. The estimated cost of the real-time control system was $7.0 million. The conclusions and the conceptual design of the real-time control system will be unaffected by an increase in system capacity for relief purposes.

3. Pollutant Loadings from Combined and Storm Sewer Discharges Study [11]

This study was carried out in order to determine the significance of the proposed real-time control system in the context of a comprehensive pollution abatement programme for the City. The scope of the study included an assessment of the effects of combined sewer overflows (controlled and uncontrolled), separated storm sewer flows, and storm flows from undeveloped areas within the City of Edmonton on the water quality of the North Saskatchewan River.

The STORM model was used to simulate flows and quality from the combined and separated sewer systems using 1969 hourly precipitation and temperature data.

Pollutant loadings from separated storm sewers and combined sewer overflows were estimated for both present and future conditions with various pollution abatement measures employed. Contributions from rural areas were considered only briefly due to lack of adequate data. The STORM model was calibrated for BOD and suspended solids using flow and quality measurements collected by the City. A simple manual model of the river water quality was used to determine the response of the river to pollutant loadings discharged from the major sources.

A comparison of the pollutant loadings discharged annually to the river under the present conditions by the sewage treatment plant (both treated effluent and overflow by-pass), combined sewer overflows, and separated storm outfalls is presented in Table 2. It is evident that the Sewage Treatment Plant (STP) is the major source of pollution in terms of BOD and coliforms. Combined sewer overflows and STP bypasses contribute about 58% of the total coliforms load discharged to the river annually. Separated areas account for 68% of the total suspended solids loads, although they account for only 20% of the total effluent.

A comparison of the effects of pollutant loads from different sources was also made for the future conditions. Five different alternatives, listed in Table 3, were compared by estimating the pollutant loads from three major sources, i.e. STP, combined and storm sewer discharges. For comparison purposes, it was assumed that bypasses at the treatment plant would not occur after completion of the ongoing expansion program in 1989. The relative effluent volumes, and the estimates of BOD, SS, and total coliforms loadings discharged annually to the river from three major sources for the five alternatives considered in this study are compared in Figures 3 to 7. Table 4 compares annual pollutant loadings to the river for 5 alternatives.

It is evident that the implementation of the proposed Real Time Control System and use of stormwater management techniques in new developments will substantially reduce the discharge of pollutant loads to the river. By stormwater management in new developments alone, a reduction of some 29% in total annual suspended loads can be achieved (see Figures 3 and 5). Total coliforms loadings to the river can be reduced up to 87% (compared to Alternative 1) by disinfecting the combined sewer overflows in addition to the implementation of the proposed in-system storage programme as suggested in Alternative 5.

A comparison of the simulation results with and without the proposed in-system storage indicates that a significant reduction in pollution will result by using the proposed storage. For example, BOD, SS and coliform loads will decrease by approximately 55%, 58% and 63% respectively, after implementation of the proposed management system. Similarly, the average annual number of overflows from combined areas, which occur mainly during the months of May to September, will drop from 24 to 8.

A simple, steady-state model based on the Streeter-Phelps approach was used to analyze the river water quality. Pollution discharged to the river was simplified by combining all separated storm sewer discharges into three major inflows to the river and by combining all combined sewer discharges into a single inflow point. Modelling results for the present conditions for BOD indicate that for a river flow of 4,000 cfs, (exceeded 50% of the time), 50% of the total increase in BOD concentration is caused by the combined sewer overflows. However, DO concentration in the river never drops below 7 mg/l. The net effect on the DO deficit of the river due to all waste sources is less than 1 mg/l within the City Limits.

River water quality analysis for the 5 alternatives described earlier was also done based on a stream flow of 4000 cfs and 1989 development and treamtent plant conditions. Results indicated that virtually no variation from existing condition is experienced with any of the 5 alternatives for the largest storm in 1969, except for total coliforms count which are slightly improved with disinfection and combined sewer in-system storage (Alternative 5). Figure 8 shows a comparison of the river water quality response for the present and future conditions at the combined sewer overflow discharge point.

The study concluded that high coliforms concentrations are a major problem in the North Saskatchewan River and the present total coliforms criterion of 5000 mpn/100 ml is frequently violated even during dry-weather conditions. It was estimated that for a typical year the total coliforms loadings from the combined and separated areas assuming treatment plant expansion as proposed resulted in 27 violations for existing conditions as compared to 13 after implementation of combined sewer in-system storage and 8 violations after disinfection along with in-system storage.

After completion of the pollution abatement study, it became apparent that water quality standards in the receiving waters of the North Saskatchewan River as set down by Alberta Environment (the Provincial environmental control authority) would not be met even with the proposed controls or even with total elimination of combined sewer overflows. Alberta Environment withdrew their restrictions and water quality standards pending further study on the limits permitted therein, and implementation of the pollution abatement program through real-time control was suspended by the City pending the outcome of their review. At present there is no indication as to when this issue may be resolved.

4. Miscellaneous Studies

A number of in-house modelling studies are being conducted by Edmonton Water and Sanitation to develop master drainage plan for the separated storm sewer systems located within the City boundaries. For instance, 8 out of 14 major storm trunk systems in the City which drain a total area of about 40,000 acres have already been analyzed using SWMM. Lumped modelling techniques are being employed to analyze these large basins. The results of these studies are being used to refine the designs for completion of the development of the respective basins and to evaluate the potential for expansion of existing drainage basins using stormwater retention techniques.

In another in-house study [12] completed in 1978, HYMO and STORM models were used to analyze flooding and water quality problems in predominantly undeveloped Mill Creek basin which has a drainage area of approximately 36.5 square miles. Hydrographs for the various design storm conditions were developed and pollutant loading from the major sources were compared.

At present, the Drainage Engineering Section is conducting a SWMM calibration and verification study using recorded rainfall and sewer flow measurements in Norwood, Fulton Drive and Groat Road areas. Preliminary study results indicate that the simulated flows compare very well with those recorded and it appears that with some calibration efforts, SWMM can be adjusted for Edmonton Conditions.

CONCLUSIONS

Successful modelling applications in the City of Edmonton have proven that large economic benefits can be derived from a substitution of the traditional approach to drainage design with more sophisticated computerized methods. Computer models are invaluable tools which provide aid in the understanding of the complex and large sewer systems and provide the engineer greater capacity and flexibility to evaluate alternatives.

ACKNOWLEDGEMENTS

I would like to acknowledge with gratitude the contribution and assistance of the following persons in developing computer modelling expertise at the City:

Dr. Paul Wisner, University of Ottawa, Ottawa, Ontario (formerly with James F. MacLaren Ltd.), who introduced computer modelling to the City of Edmonton and contributed in many studies for the City.
R.S. Cebryk, Cumming-Cockburn & Associates Limited (formerly with the City of Edmonton).
J.F. Hugo, Chief - Drainage Engineering, City of Edmonton.

TABLE 2

TOTAL AVERAGE ANNUAL

LOADS TO THE RIVER

	STP	STP	Storm Sewer Out-flows	Combined Sewer Overflows	Total
Volume (10^9 cu. ft.)	2.79	0.14	0.75	0.11	3.79
Suspended Solids (10^6 lbs.)	6.03	2.34	24.0	2.8	35.17
BOD (10^6 lbs.)	5.02	1.46	1.12	0.41	8.01
Total Coliforms (10^{17} MPN)	11.1	8.57	0.6	7.6	27.87

TABLE 4

COMPARISON OF ANNUAL LOADINGS

TO THE RIVER FOR FIVE ALTERNATIVES

	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5
Volume (10^9 cu. ft.)	5.2	5.1	5.2	5.1	5.1
Suspended Solids (10^6 lbs.)	43.3	41.7	30.6	29.0	41.7
BOD (10^6 lbs.)	9.5	9.3	9.2	9.0	9.3
Total Coliform (10^{15} MPN)	841.2	362.0	841.2	362.0	110.0

TABLE 3

LIST OF ALTERNATIVES

Alternative 1

1989 treatment plant effluent conditions.

Existing combined sewer overflows without in-system storage.

Separated areas without stormwater management.

Alternative 2

1989 treatment plant effluent conditions.

Proposed in-system storage in combined areas.

Separated areas without stormwater management.

Alternative 3

1989 treatment plant effluent conditions.

Existing combined sewer overflows without in-system storage.

Separated areas with stormwater management in new developments.

Alternative 4

1989 treatment plant effluent conditions.

Proposed in-system storage in combined areas.

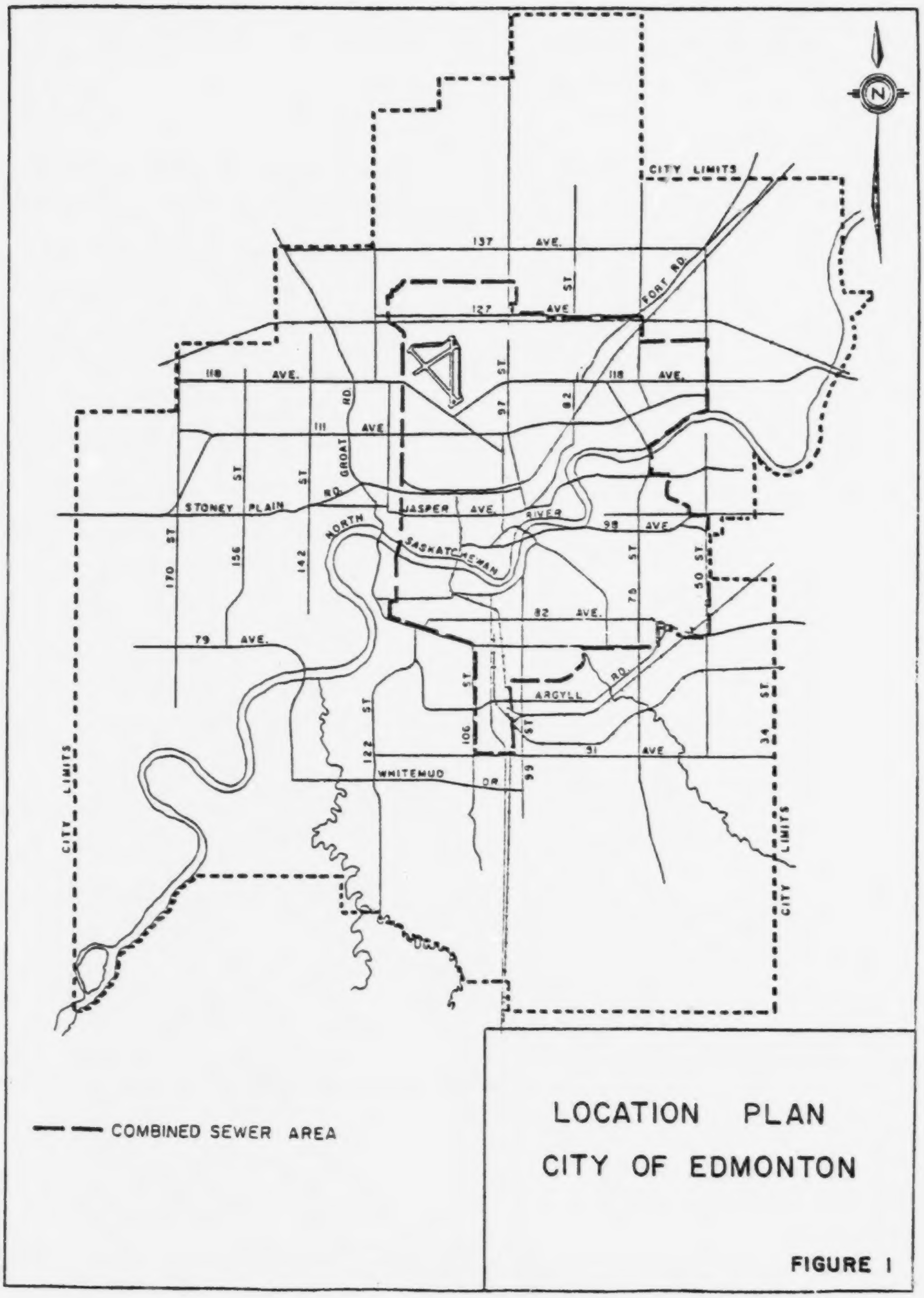
Separated areas with stormwater management in new developments.

Alternative 5

1989 treatment plant effluent conditions.

Proposed in-system storage in combined areas plus disinfection of overflows.

Separated areas without stormwater management.


CITY LIMITS
137 AVE.
127 AVE
118 AVE.
111 AVE
FORT RD.
STONEY PLAIN RD.
GROAT RD.
JASPER AVE.
NORTH SASKATCHEWAN RIVER
98 AVE
82 AVE.
79 AVE.
ARGYLL RD.
91 AVE
WHITEMUD DR
170 ST
156 ST
142 ST
122 ST
106 ST
99 ST
97 ST
82 ST
75 ST
50 ST
34 ST
N
COMBINED SEWER AREA

LOCATION PLAN
CITY OF EDMONTON

FIGURE 1

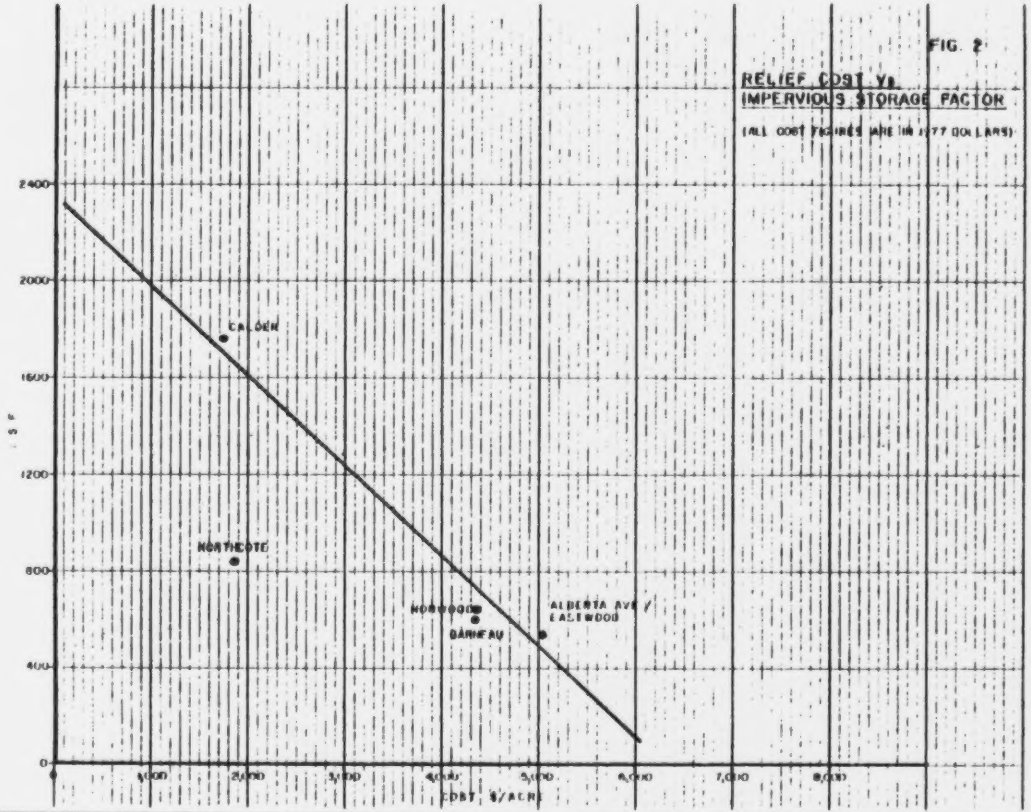


FIG. 2

RELIEF COST Vs. IMPERVIOUS STORAGE FACTOR

(ALL COST FIGURES ARE IN 1977 DOLLARS)

COMPARISON OF POLLUTION FROM TREATMENT PLANT AND SEWER OVERFLOWS

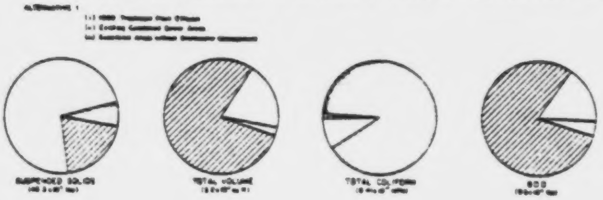


FIG. 3

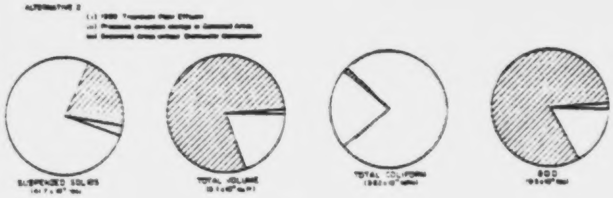


FIG. 4

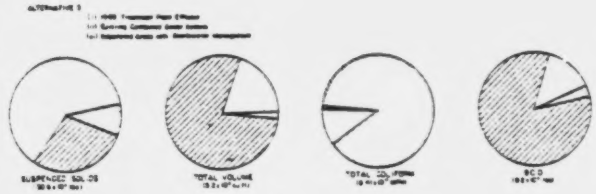


FIG. 5

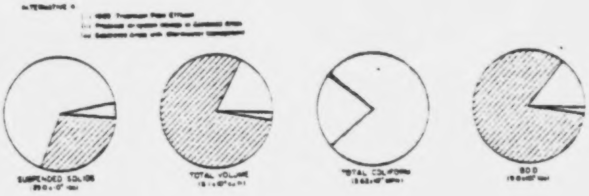


FIG. 6

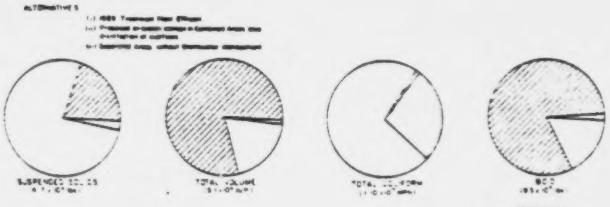
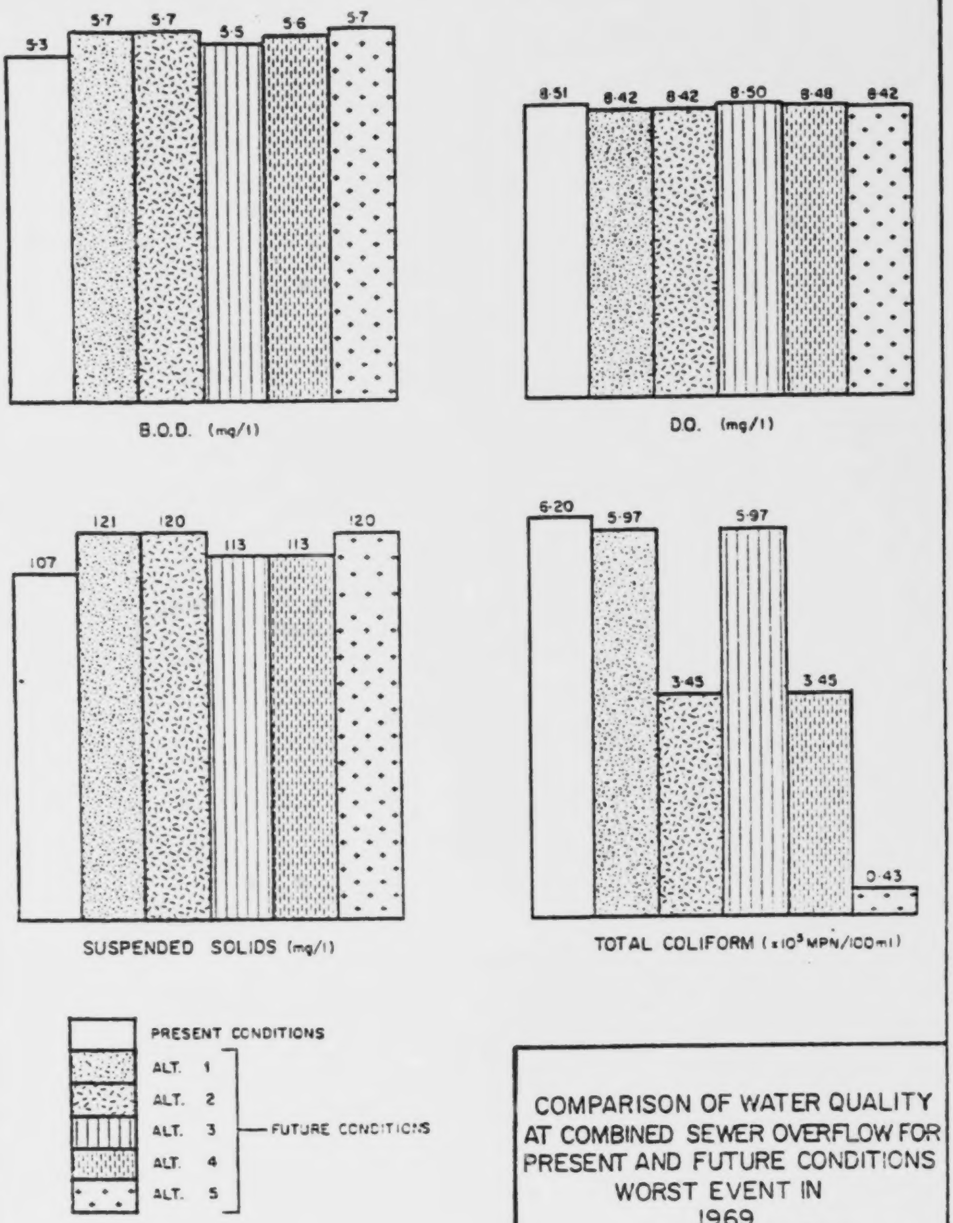


FIG. 7

STP ▨
SEP ☐
COMB ☐

TOTAL ANNUAL AVERAGE LOAD TO RECEIVING WATER FOR FUTURE CONDITIONS

Reproduced from best available copy.



COMPARISON OF WATER QUALITY
AT COMBINED SEWER OVERFLOW FOR
PRESENT AND FUTURE CONDITIONS
WORST EVENT IN
1969

Figure 8

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STORMWATER RUNOFF MODELING OF THE TAMPA PALMS PROPERTY

By

Kevin Smolenyak[1]

Introduction

The Tampa Palms development which is located approximately eleven miles northeast of downtown Tampa is a residentially oriented master planned community totaling 5,400 acres in size with an ultimate density of 13,000 units. Stormwater discharges from this property will enter the Hillsborough River, a potable source of water for the City of Tampa.

In order to assess the effects of the development of the Tampa Palms Property on the quantity and quality of stormwater runoff entering the receiving systems, the Storage Treatment Overflow Runoff Model (STORM)[1, 2] was employed. STORM, a widely used and proven hydrologic model (continuous simulation), was initially applied to the project in its current undeveloped condition, using water quantity and quality data collected from several monitoring stations strategically located at surface runoff sites throughout the property for model calibration (Figure 1). The proposed changes in land use and drainage characteristics due to development were then incorporated into the model to predict future stormwater runoff quantity and quality conditions and to analyze potential impacts to receiving systems.

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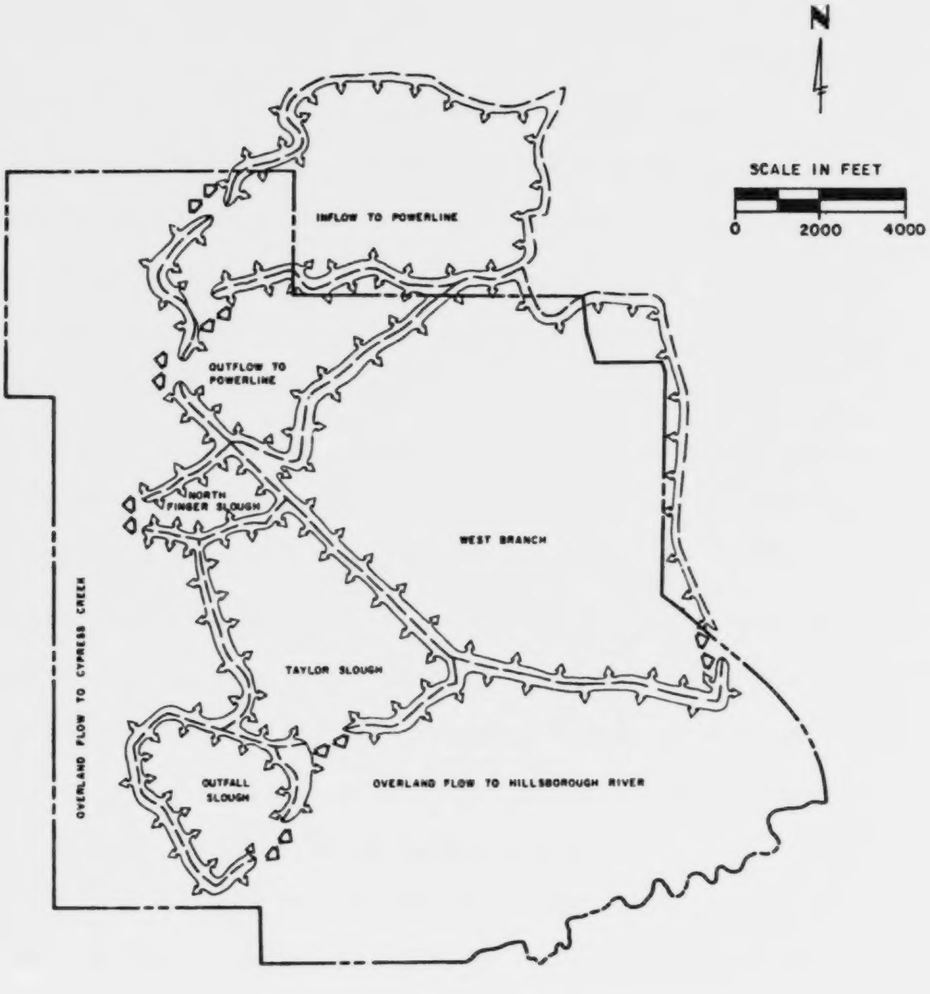
Figure 1
Pre-Development Drainage Basins


N
SCALE IN FEET
0
2000
4000
INFLOW TO POWERLINE
OUTFLOW TO POWERLINE
NORTH FINGER SLOUGH
WEST BRANCH
OVERLAND FLOW TO CYPRESS CREEK
TAYLOR SLOUGH
OUTFALL SLOUGH
OVERLAND FLOW TO HILLSBOROUGH RIVER

Model Input Requirements

The STORM model requires hourly precipitation data for simulation purposes. Accordingly, hourly precipitation data for the years 1948-1975 were obtained from the U.S. National Weather Service (Ashville, North Carolina) for the Tampa Airport station, which is approximately thirteen miles southwest of the Tampa Palms property. In addition, continuous (hourly) rainfall data from a rain gage at the City of Tampa Water Treatment Plant adjacent to the property was used in the analysis.

Average daily pan evaporation rates for each month of the year are used for the recovery of depression storage and soil moisture capacity. Fourteen years of pan evaporation data from the Lake Alfred experimental station (closest available long-term record) were analyzed to determine average daily pan evaporation rates.

An extensive literature review was undertaken to determine appropriate average daily pollutant accumulation rates for each land use in the developed and undeveloped areas. It was determined that studies conducted by the United States Geological Survey (USGS) in Broward County, Florida[3, 4, 5, 6, 7,] and Miami, Florida[8] would provide the most suitable loading rates for modeling the developed areas of the Tampa Palms property for the following reasons.

1. The USGS study areas and Hillsborough County are similar geographically and climatically (e.g. mild ground slopes, similar rainfall patterns etc.)
2. The published USGS stormwater pollutant loadings were the result of an excellent water quality monitoring program[8, 9] (e.g. well documented study, an extensive number of storms

monitored, a large number of water quality samples taken for each storm resulting in well defined pollutographs, good data collection and data management system used).

3. Four land use sites were monitored in the study (i.e. single family residential, transportation, commercial shopping center for the Broward County sites and a multi-family residential site in Miami). These land uses encompass the various types of urban development planned for the Tampa Palms property.
4. The single family residential site in Broward County employs open channel swale drainage which is planned for the Tampa Palms site.

Loading rates used for the undeveloped areas were average values obtained from studies within Florida.[10]

Basins Modeled

Within the Tampa Palms property six basins were defined and surface water monitoring stations setup at each (Figure 1). Of these six basins Taylor Slough (MLS) and Outfall Slough (CPS) were modeled by STORM for the following reasons.

1. Each basin lies entirely within the Tampa Palms property.
2. The boundaries of each basin are well defined.
3. The channels draining these basins are well defined.
4. The Taylor Slough Basin boundaries will only change slightly after development. Therefore, the basin is ideal for comparing stormwater quantity and quality before and after development.
5. The Outfall Slough basin (pre-development) serves as the existing subbasin of a much larger post-development Outfall Slough basin.

Since high density housing is planned for this subbasin, the magnitude of the effects of such development on stormwater quality is of interest.

6. The other post-development basin, West Branch, would be difficult to model in pre and post-development phases because of the discharge of backwash water from the Tampa Water Treatment Plant onto the basin. In addition, accurate comparisons of the pre and post-development stormwater quality is not possible because the two basins are different in size and boundaries.

For modeling purposes the pre and post-development Taylor Slough and Outfall Slough basins were subdivided into land uses. A comparison of the land uses of the basins is given in Table 1.

Calibration of Pre-Development Basins

Certain input parameters in the STORM model are highly site specific and affect such factors as the quantity and time distribution of stormwater runoff and the magnitudes of the pollutants modeled. Calibration involved the adjustment of these input parameters until reasonable agreement was achieved between predicted and measured (or actual) stormwater quantity and quality. Considerable effort was expended in determining appropriate values for the various parameters required as input for the model to help insure the validity of the output.

Water quantity and quality information obtained from the surface water monitoring stations at both Taylor and Outfall Sloughs were used to calibrate the model for each pre-development basin. The pollutants sampled at these sites were modeled by STORM and include BOD_5, TSS

Table 1. Land Use Breakdown (Taylor and Outfall Basins)
Taylor Slough - Pre and Post-Development

Land Uses

Taylor Slough/Pre-Development

Land Use	Area (Acres)	% of Basin Area
Rangeland	188.3	39.0
Wetland	162.8	33.7
Upland Forest	131.9	27.3
	Total=483.0	

Taylor Slough/Post-Development

Land Use	Area (Acres)	% of Basin Area
Single Family Residential	294.7	52.3
Open Space	126.2	22.4
Golf Courses	64.3	11.4
Roads	54.7	9.7
Commercial	23.7	4.2
	Total=563.6	

Outfall Slough - Pre and Post-Development

Land Uses

Outfall Slough/Pre-Development

Land Use	Area (Acres)	% of Basin Area
Wetland	121.0	56.8
Rangeland	70.5	33.1
Upland Forest	21.5	10.1
	Total=213.0	

Outfall Slough/Post Development (Subbasin No. 5)

Land Use	Area (Acres)	% of Basin Area
Open Space	141.0	60.9
Multi-Family Residential	75.4	32.6
Roads	10.0	4.3
Commercial	5.1	2.2
	Total=231.5	

(total suspended solids), TOTN (total nitrogen as N), TOP (total orthophosphate as P and TCOL (total coliforms).

Hourly rainfall data (September, 1978 thru June, 1979) obtained from a station at the City of Tampa Water Treatment Plant, which lies adjacent to the Tampa Palms property were used in the model simulation. Calibration procedures were conducted using information gathered between December, 1978 and May, 1979 at the appropriate water monitoring stations For each pre-development basin the models were calibrated for stormwater runoff quantity followed by calibration for runoff quality.

For the calibration of stormwater runoff quantity reasonable agreement was achieved between:

1. Total runoff volumes for the period of record.
2. Individual event runoff volumes.
3. Peak runoff rates, runoff durations and timing (i.e. the shape of the individual hydrograph).

For the calibration of stormwater runoff quality average annual pollutant concentrations, determined using pollutant accumulation rates based on water quality studies in Florida[10], were compared to the mean concentrations determined via the surface water monitoring program. In the STORM model a linear relationship exists between the pollutant accumulation rates and output pollutant loads and concentrations.

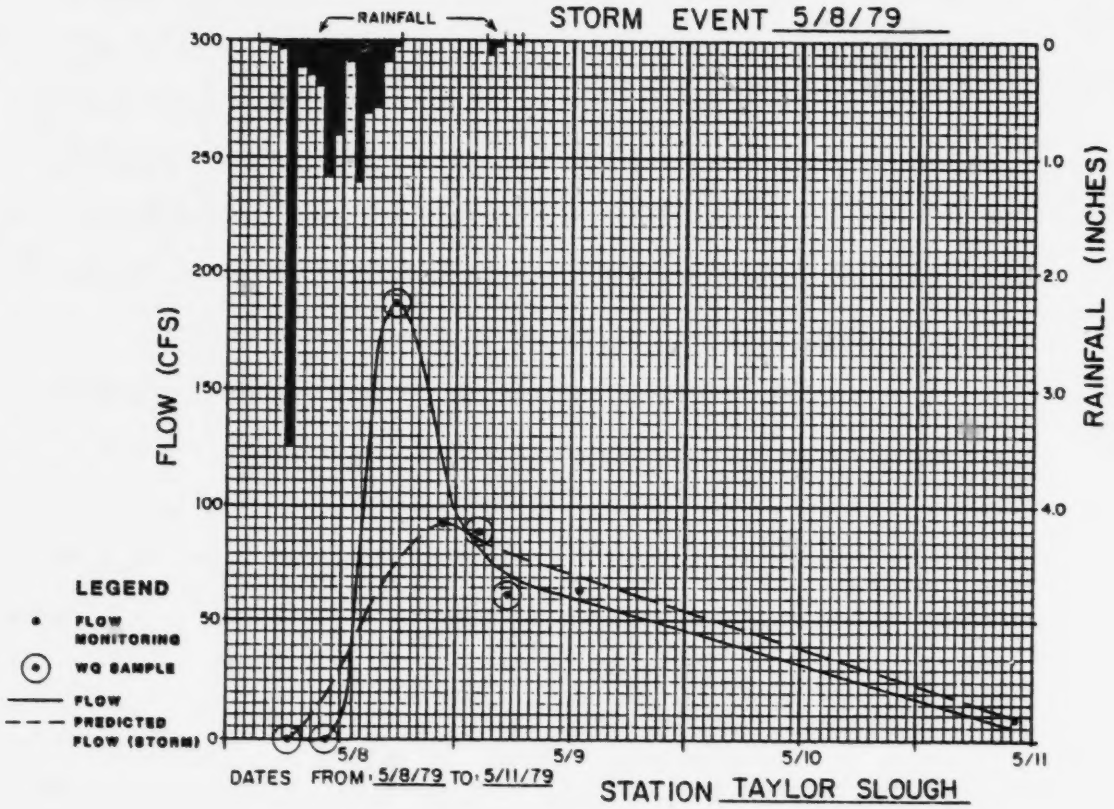
Accordingly, appropriate changes were made in the loading rates for the individual pollutants to achieve agreement between the measured and predicted pollutant concentrations.

Calibration Results-Quantity

Good agreement was achieved between measured and predicted runoff volumes and individual storm volumes for Taylor Slough. Individual storm eventsexcept that of May 8, 1979 (Figure 2) were calibrated by comparing mean daily runoff discharges. Drainage within the basin is slow and storm event durations are on the order of days, therefore the use of mean daily discharges for calibration is sufficient. Figure 2 represents the measured and calibrated outflow hydrograph for Taylor Slough for a 10.6 inch rainfall (10.25 inch, 12 hour total = 100 year event). Measured and calibrated runoff volumes for this event were 7.47 inches and 7.88 inches respectively. The discrepancy in peak flows can be attributed to the extreme nature of the event, the complex nature of the basin and the influence of the initial soil moisture conditions chosen for the relatively short simulation period.

The analysis of runoff volume for Outfall Slough indicated a high baseflow component within the basin. Rainfall infiltrates into the well drained soils and is transported laterally through the soils due to a steep groundwater gradient to the outfall point of the basin. Examination of groundwater wells within the basin indicates that between the uplands area in the northwest portion of the basin and the outfall point, a distance of 4,500 feet, a 9.3 foot gradient exists during the wet season with a slightly higher gradient during the dry season. STORM does not simulate base flow. Water quality modeling, however, should not be significantly affected since the major source

Figure 2
Taylor Slough - Runoff Hydrograph for
May 8, 1979 Storm Event



of pollutant loads results from surface runoff. Pollutant contributions from base flow are minor due to filtering capabilities of the soils and the slow movement of water through the soils. Water quality samples were also taken at Outfall Slough for the 10.6 inch rainfall (10.25 inch, 12 hour total=100 year event) which began May 8, 1979. Backwater from Cypress Creek inhibited flow out of the basin approximately two days after initial stormwater runoff from the basin.

Comparisons of calibrated values for the soil storage parameters within the STORM model to those of a study by the USGS[11] using a modified version of the Georgia Tech watershed model indicate excellent agreement. The USGS study was conducted to simulate streamflow in the Hillsborough River. The Tampa Palms property lies within the 160 square mile basin modeled by the USGS.

Calibration Results-Quality

As mentioned earlier quantity calibration for both Taylor and Outfall Slough involved altering pollutant accumulation rates in order that measured and predicted (annual) mean pollutant concentrations were identical.

Comparisons of individual storm pollutant loadings were only possible for the May 8, 1979 event. Measured pollutant loads (determined using water quality samples taken at various times during the storm (Figure 2) are compared with those predicted by STORM below.

Taylor Slough - Storm Event (5/8/79) Total Loads (Pounds)

	BOD	TSS	TOP	TNIT	TCOL (MPN)
Measured	1618	3235	33.5	184	74×10^{12}
Model	630	4347	8.8	163	28×10^{12}

Outfall Slough - Storm Event (5/8/79) Total Loads (Pounds)

	BOD	TSS	TOP	TNIT	TCOL (MPN)
Measured	844	1628	6.35	171	31×10^{12}
Model	257	1653	2.49	39	6.2×10^{12}

STORM Application to Post-Development Basin

After calibrating both Taylor and Outfall Sloughs using hourly rainfall data from a station at the City of Tampa Water Treatment Plant, the two basins were modeled using hourly rainfall data from the National Weather Service (NWS) station at the Tampa Airport (nearest long-term hourly rainfall record available). The rainfall record chosen for simulation purposes (6/48 - 8/52, a total of 1553 days) represents a period of average rainfall for the Tampa area. The results of these simulations will be discussed later when they will be compared with the results for the post-development basins for the same period of record.

Calibrated input variables obtained from STORM runs of the pre-development basins were applied to the STORM runs of the post-development basins when appropriate. For example, input parameters affecting hydrograph shapes were changed to account for the decrease in the time of response of the system (i.e. decreased time of concentration and time of recession over time to peak).

Pollutant accumulation rates for single family residential, commercial and highway (road) areas were derived from extensive stormwater studies conducted by the USGS in Broward County, Florida[3, 4, 5, 6, 7] Multi-family rates were derived from a similar USGS study in Miami, Florida.[8] For the open space areas calibrated accumulation rates, determined from pre-development STORM simulations for the Woodlands/Wetlands areas, were used. Golf courses were assigned the calibrated accumulation rates for pasture areas to account for increased nitrogen and phosphorous loadings due to fertilization.

Results and Discussion

Tables 2 and 3 summarize the results of the STORM simulations (6/48 - 8/52) for the pre and post-development Taylor Slough and Outfall Slough basins. The following findings are noteworthy:

1. The number of storm events for the simulation period (6/48 - 8/52) increases after development. This is due to changes in the shapes and durations of individual storm hydrographs following development. In addition, response times in the developed basins are shorter and therefore runoff defined as a single event (an event is defined as beginning when runoff is initiated and ending when runoff stops in the pre-development basin might be characterized as two separate events in the post-development basin.
2. For the pre-development basins 16.0% of the rainfall appeared as surface runoff. A USGS study,[11] using 10 years of data (10/64 - 9/74) from a streamflow station at Cypress Creek near Sulfur Springs, reports a mean annual runoff coefficient of

Table 2. Taylor Slough - A Comparison of Pre and Post-Development Water Quantity and Quality Conditions for the Simulation Period June, 1948 through August, 1952.

Taylor Slough/Pre-Development

Number of storm events = 87
Rainfall = 207.34 inches (period of record 6/48-8/52)
Runoff = 33.25 inches
Fraction of rainfall as runoff = 16.0%
Area of basin = 483 acres

Average Annual Pollutant Loads

	BOD_5	TSS	TOP	TOTN	TCOL
Total Pounds	2,210	13,547	31	570	97×10^{12} MPN
Pounds/acre	4.58	28.1	.064	1.18	2×10^{11} MPN/acre
Conc (mg/l)	3.05	18.7	.043	.79	2.95×10^{4} MPN/100 ml

Taylor Slough/Post-Development

Number of storm events = 112
Rainfall = 207.34 inches (Period of record 6/48-8/52)
Runoff = 26.75 inches
Fraction of runoff = 12.9%
Area of basin - 563.7 acres

Average Annual Pollutant Loads

	BOD_5	TSS	TOP	TOTN	TCOL
Total Pounds	5,513	20,110	67	1,074	324×10^{12} MPN
Pounds/acre	9.78	35.7	.119	1.91	5.75×10^{11} MPN/acre
Conc (mg/l)	8.06	29.4	.097	1.57	10.5×10^{4} MPN/100 ml
% Increase over Pre-development loads	114	27	86	62	188

Table 3. Outfall Slough - A Comparison of Pre and Post-Development Water Quantity and Quality Conditions for the Simulation Period June, 1948 through August, 1952.

Outfall Slough/Pre-Development

Number of storm events = 91
Rainfall = 207.34 inches (Period of record 6/48-8/52)
Runoff = 33.05 inches
Fraction of rainfall as runoff = 16.0%
Area = 213 acres

Average Annual Pollutant Loads

	TSS	BOD_5	TOP	TOTN	TCOC
Total Pounds	5,262	911	8.4	237	39.3×10^{12} MPN
Pounds/acre	24.7	4.28	.04	1.11	1.8×10^{11} MPN/acre
Conc (mg/l)	16.5	2.86	.03	.74	27,200 MPN/100 ml

Outfall Slough (Subbasin No. 5)/Post-Development

Number of storm events = 94
Rainfall = 207.34 inches (Period of record 6/48-8/52)
Runoff = 27.1 inches
Fraction of rainfall as runoff = 13.0%
Area = 231.5 acres

Average Annual Pollutant Loads

	BOD_5	TSS	TOP	TOTN	TCOC
Total Pounds	3,054	102,748	55.9	1124	172×10^{12} MPN
Pounds/acre	13.2	444	.24	4.86	7.4×10^{11} MPN/acre
Conc (mg/l)	10.8	363	.20	3.96	133,455 MPN/100 ml
% Increase over Pre-development loads	152	1,700	400	338	311

16.6%. The drainage basin studied by the USGS is 160 square miles in area and includes the Tampa Palms property.

3. Simulations indicate that surface runoff will decrease by approximately 20% following development. This reduction is primarily attributed to the improvement of soil hydrologic ratings from pre to post-development conditions as a function of the enhanced setting resulting from drainage network implementation. Additional reduction in runoff is expected to occur as a result of the drainage system's increased potential for groundwater recharge and evapotranspiration.

4. Without accounting for the significant stormwater treatment which will be achieved in the post-development, Tampa Palms water management system simulations indicate stormwater pollutant loads (lb/acre/year) at the outfall points of the basins will increase following development. The larger increases in pollutant loadings for Outfall Slough can be attributed to the high pollutant accumulation rates used in modeling the high density multi-family residential area found in the post-development Outfall basin.

The predicted increases in stormwater pollutant loads due to the development of Taylor Slough and Outfall Sloughs are highly conservative (i.e. predicted increases in pollutant loads are significantly higher than expected). The above conclusion is based upon the fact that pollutant removals via stormwater treatment within the post-development drainage system (i.e. treatment due to retention of stormwater in DRA's) and detention throughout the system cannot be modeled by STORM. This treatment can only be reflected by decreasing the pollutant accumulation rates, which are linearly related to the stormwater runoff quality. As mentioned earlier, in the post-development basins pollutant accumulation rates for single and multifamily residential sites, commercial sites and highway sites were developed from extensive stormwater runoff studies conducted by the USGS in Broward County[3, 4, 5, 6, 7] and Miami, Florida[8]. In these studies pollutant loads were measured at outfalls near the source allowing for little or no stormwater detention. Only the single family site in Broward County, Florida provided any type of treatment before water quality samples were taken. This treatment, which can be considered minimal compared to that which will be obtained in the Tampa Palms drainage system, consisted of drainage to grass swales which discharged directly into a storm sewer where the water quality samples were gathered.

Significant stormwater treatment will be obtained when runoff from impervious areas enters into the post-development drainage system of the Tampa Palms property. This drainage system is comprised of grass swales (roadside and rear yard) which convey stormwater overland to a system of drainage retention areas (natural wetlands) and lakes followed by final diffuse sheet flow outfall to naturally vegetated wetlands prior to introduction to receiving waters. The processes governing these treatments include grass-soil filtration, nutrient uptake via cypress roots and underlying sediments, controlled sedimentation in drainage retention areas and emergent macrophytic uptake in the shallow littoral zone of the lakes. None of these removals were accounted for in the post-development basin simulations (i.e. unadjusted loading rates developed from the USGS studes were used).

The actual pollutant removals which can be attained in the post-development drainage system are difficult to quantify. However, Heaney and Huber[12, 13] have developed a relationship between pollutant removals and stormwater detention times based on studies of the Kissimmee River basin in Florida. Applying this relationship to the post-development drainage basins leads to the following treatment efficiencies.

Basin	Runoff Detention (inches)	Detention time (days)	Maximum % Pollutant Removal
Taylor Slough	1.03	16	86
Outfall Slough	2.81	24	95
West Branch	1.47	30	98

The reported detention times assume that the drainage system is not overloaded (i.e. individual basin storage is capable of handling the amount of stormwater via slow release through discharge pipes without overflows of the discharge weirs). Overflows (high rate discharges) would lead to reduced detention times and corresponding decreased pollutant removals. However, STORM simulations (period of rainfall 6/48 - 8/52) of the post-development Taylor and Outfall Sloughs, assuming a single conservative drain-down rate for each detention basin, indicate that discharges over the weirs are infrequent, thereby satisfying the assumption criteria.

Applying these removals to the STORM simulation results for the post-development Taylor Slough and Outfall Slough (Subbasin No. 5) basins, the following results are obtained (See Table 4).

The reported removals apply to treatment within the basins. Further removals are possible as the stormwater runoff discharges as sheetflow across naturally vegetated wetlands (Hillsborough River Conservation Area) prior to introduction to receiving waters (Hillsborough River).

The net impacts of stormwater discharges from the Tampa Palms property on the Hillsborough River should be minimal, because pollutant loads (mass) from the developed property will only constitute a small percentage of the loads carried by the river. Comparisons of calculated yearly pollutant loads for the post-development Taylor and Outfall (Subbasin No. 5) basins and the Hillsborough River at Fowler Avenue (approximately 1.5 miles south of the property) are shown on Table 5. Loads for the Tampa Palms property were obtained from STORM simulations (6/48 - 8/52) and are reported for prior to treatment and after treatment conditions (See previous discussion). Calculated yearly loads for the Hillsborough River at Fowler Avenue are based on water quality studies by Hatcher and Courtney[14] and a conservative average discharge of 450 cfs.

For all the pollutants, total loads for the post-development basins represent a minor to very minor percentage of the total loads for the Hillsborough River. Impacts, therefore are expected to be minimal. In addition, stormwater discharges from the developed drainage system will be attenuated (via controlled release) which will increase the ability of the Hillsborough River to assimilate the loads.

Table 4

Average Annual Pollutant Loads (Pounds/Acre) For Taylor and Outfall Sloughs (Simulation Period June, 1948 through August, 1952)

	BOD_5	TSS	TOP	TOTN	TCOL
Taylor Slough (Pre-Development)	4.58	28.1	0.064	1.18	$2x10^{11}$ MPN/Acre
Taylor Slough (Post-Development prior to treatment within Tampa Palms Water Management System)	9.78	35.7	.119	1.91	$5.75x10^{11}$ MPN/Acre
% Increase over Pre-Development Loads	(114)	(27)	(86)	(62)	(188)
Taylor Slough (Post-Development following treatment within Tampa Palms Water Management System)	2.62	12.2	.033	.60	$1.37x10^{11}$ MPN/Acre
% Increase over Pre-Development Loads	(-43)	(-57)	(-48)	(-49)	(-31)
Outfall Slough (Pre-Development)	4.28	24.7	.04	1.11	$1.8x10^{11}$ MPN/Acre
Outfall Slough (Post-Development prior to treatment within Tampa Palms Water Management System)	13.2	444	.24	4.86	$7.4x10^{11}$ MPN/Acre
% Increase over Pre-Development Loads	(152)	(1700)	(400)	(338)	(311)
Outfall Slough (Post-Development following treatment within Tampa Palms Water Management System)	2.38	26.0	.03	.77	$1.5x10^{11}$ MPN/Acre
% Increase over Pre-Development Loads	(-44)	(+5)	(-25)	(-31)	(-17)

Table 5

A Comparison of Average Annual Pollutant Loads (lbs)
For the Hillsborough River at Fowler Avenue, Taylor Slough and Outfall Slough

LOCATION	% of Total Basin Area	BOD_5		TOTN		TOP		TCOL	
		Load	%	Load	%	Load	%	Load	%
Hillsborough River at Fowler Avenue	100	$7.96x10^5$	-	$7.53x10^5$	-	$2.80x10^5$	-	$6.18x10^{15}$	-
Taylor Slough (prior to treatment)	.14	5510	.69	1070	.14	67	0.24	$3.24x10^{14}$	5.2
Taylor Slough (after treatment)	.14	1480	.19	338	.04	19	.007	$7.72x10^{13}$	1.3
Outfall Slough (subbasin No. 5 prior to treatment)	.06	3050	.38	1120	.15	56	.02	$1.72x10^{14}$	2.8
Outfall Slough (subbasin No. 5 after treatment)	.06	551	.07	178	.024	6.9	.002	$3.5x10^{13}$	.57

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Drainage System Design and Analysis of the Tampa Palms Property

By

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Introduction

This paper provides a review of the design objectives and analysis procedures as well as a summary of selected simulation results for an investigation of the drainage system proposed for the Tampa Palms property, Hillsborough County, Florida. The proposed drainage system is of current interest since it incorporates numerous "non-structural" Best Management Practices including the use, as detention and conveyance networks, of natural and man-made wetland systems. In addition, a simple procedure developed by the author referred to as the Conservative Weir Approach, employing the Soil Conservation Service unit hydrograph method and the TR-20 Hydrology Computer program was used in the design/analysis investigations and is reported on herein.

Whereas such investigations are considered only as preliminary studies in regards to structure design, they have proven to be of substantial value in obtaining regulatory agency acceptance. In addition, the results have been used as input data and as qualitative support for continuous water quality modeling (STORM) where simulation results require detention area relationships of stage and discharge.

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General Project Description

The study site is located in northwestern Hillsborough County, Florida, approximately 12 miles from downtown Tampa. The site is bounded on the west by Cypress Creek and its floodplain; on the south by Cypress Creek and Hillsborough River and their respective floodplains; and on the east by Levee 112N (Lower Hillsborough Flood Detention Area) which lies west of and adjacent to Trout Creek. An area of about 5400 acres included within the property boundaries is owned by Deltona.

Existing vegetative cover has been described pursuant to Florida Land Use and Classification System by DuBois and Courtney and soil mapping of the property has been performed through the Hillsborough County Extension - Soil Conservation Service. Topography for the study area has been reported through the auspices of the Southwest Florida Water Management District (SWFWMD).

A review of climatological data and summaries provided through the National Oceanic and Atmospheric Administration reveal several salient facts regarding the study vicinity. Firstly, the area is subject to frequent thundershowers during the months of June through September. During this period about 60 percent of the annual rainfall can be anticipated.

Average annual rainfall for the Tampa station for 79 years of record is 47.72 inches. The wettest year recorded was 1959 with 76.57 inches of precipitation and the driest year occurred in 1956 when 28.89 inches was recorded. An evaluation of recent rainfall for Tampa indicates an upturn trend in 16 year moving average rainfalls after continual decreases for the past 9 years. Finally, annual evaporation as measured at the Lake Alfred Experimental Station (closest established site) is reported to be 68.74 inches based on 13 years of data.

Drainage System Design/Performance Objectives

Meaningful storm water drainage design addresses both the quantity and quality aspects of excess precipitation. In particular, of paramount concerns with the quantity of discharge are the protection of the project against flooding, and the attenuation of peak runoff rates and the reduction of total volumes from within the developed property to approximate natural discharge conditions and thus prevent flooding and erosion problems downstream, or upstream as in the case of backwater flooding. The quality of the drainage discharge must be such that it will be compatible with basin management objectives and regulatory requirements and not cause significant degradation to intermediate receiving bodies.

In order to most efficiently and cost effectively meet the quantity and quality objectives, natural drainage systems of depressions and sloughs and the accompanying natural vegetation should be maintained and utilized to the fullest extent practical. This will result in a system that is not only visually pleasing and effective in meeting design objectives, but one which has low initial costs and requires a minimum of skilled maintenance for upkeep. It has been the goal to design such a system for Tampa Palms and the approach and results of those design efforts are herein presented.

Drainage

Numerous considerations and objectives have been incorporated into the design of the subject project drainage system. The following list includes, not necessarily in order of importance, several of those items considered in the drainage design process:

1. Provide adequate drainage to ensure protection to private property and drainage works in the development.
2. Design system to prevent flooding impacts on upstream and downstream properties.
3. Design system to minimize fill requirements thereby preserving natural setting and reducing development costs (energy and resources).
4. Establish sub-basin control water levels to be consistent with recorded pre-development low ground water table levels.
5. Maintain surficial ground water table regime to insure continued ground water recharge contribution to Cypress Creek and Hillsborough River.
6. Incorporate and preserve function and hydro-period of wetland areas included in development plan.
7. Preserve existing drainage basins and patterns of flow to the greatest degree possible.
8. Maximize system's storm runoff retention and detention capabilities.
9. Attenuate peak discharges and total quantities to approximate pre-development conditions.
10. Provide a system that is consistent with design standards and minimizes the need for maintenance and/or operational procedures.
11. Insure that discharge velocities will not result in scour of contributory or receiving systems.
12. Provide for maximum use of grassed conveyance swales and non-structural controls in lieu of storm sewer systems.
13. Allow for low flow discharge (approximating natural conditions) at two locations through the proposed Cypress Creek levee to be conveyed

via existing natural outfalls at times when Creek stage is low.

14. Insure that the ultimate outfalls of the drainage sub-basins are spatially separated from the open water of the receiving body by a wetland marsh "buffer" system.
15. Incorporate as many best management techniques (BMT's) as possible into the drainage designs so as to reduce impacts (quality and quantity) of project on receiving water bodies.
16. Maintain and provide for "base flow" discharges from the property to replenish receiving bodies during low flow conditions.

Design Concepts

The major obstacle to designing the drainage system for this project was not the ability of the system to adequately drain the site of excess rainfall. Sufficient topographic relief and existing hydrologic conditions are more than adequate to allow for a standard design to meet local flooding objectives. The burden of the task proved to be a result of the ambitious determination to incorporate and maintain natural wetland systems into the drainage plan while maintaining the hydraulic viability of the conveyance network so as to prevent both internal and external flooding. As a result of experiences in similar situations, review of the literature, and discussions with knowledgeable agency personnel, the design concepts necessary to achieve the design/performance objectives were able to be defined.

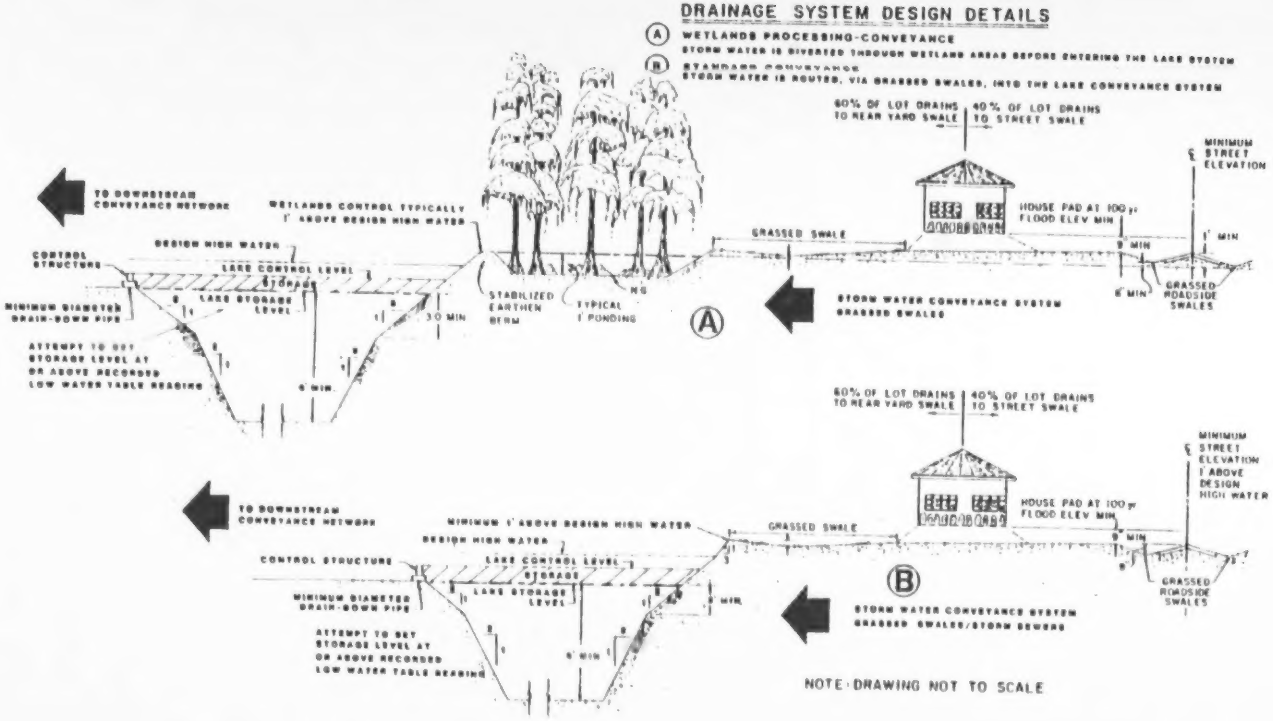
The proposed drainage system can best be described as a network of grassy swales, natural wetland areas and lakes which have the potential to store, treat and, subsequently, slowly release excess rainfall from the site to naturally vegetated sheet flow areas prior to introduction to receiving waters. The system is designed to be positively drained (i.e., by gravity) and will not be adversely impacted by receiving body flood stages up to and including the 25 year flood. Beyond that, receiving water flood stage, localized internal flooding (minimum elevation streets) of short duration (as a result of secondary drainage structure inadequacies) could occur depending upon the severity of the accompanying storm event. However, the statistical likelihood of simultaneous flood and design storms is highly remote and standard engineering practice and financial practicality do not support the investigation and design for such extreme events. We would, however, note that the system could still positively drain rainfall from the property even in the case of the 100 year flood in the receiving body.

Stormwater within the drainage system is generally conveyed from upland development sites to wetland areas, to lakes and other conveyance ways. In all instances where the wetlands are part of the conveyanceway the system is designed so that velocity of flows will be low enough to prevent scour or other damage to it.

The preserved wetland areas within the drainage retention areas (DRA's) will be allowed to maintain their natural hydroperiod by means of either stabilized earthen berms or control structures (weirs) which will prevent severe unnatural water level draw-downs.

To aid the reader in understanding the concepts described above, attention is directed to Figure 1. The typical flow of runoff from the system is portrayed as traveling first through roadside or rearyard swales, thence into wetlands or lakes, and then through the storm water conveyance system to the ultimate outfall. From the point of ultimate outfall storm water flows in an overland fashion through extensive areas of natural

Figure 1
Drainage Design Concepts


DRAINAGE SYSTEM DESIGN DETAILS
Ⓐ WETLANDS PROCESSING-CONVEYANCE
STORM WATER IS DIVERTED THROUGH WETLAND AREAS BEFORE ENTERING THE LAKE SYSTEM
Ⓑ STANDARD CONVEYANCE
STORM WATER IS ROUTED, VIA GRASSED SWALES, INTO THE LAKE CONVEYANCE SYSTEM
60% OF LOT DRAINS TO REAR YARD SWALE
40% OF LOT DRAINS TO STREET SWALE
MINIMUM STREET ELEVATION
TO DOWNSTREAM CONVEYANCE NETWORK
WETLANDS CONTROL TYPICALLY 1' ABOVE DESIGN HIGH WATER
DESIGN HIGH WATER
CONTROL STRUCTURE
LAKE CONTROL LEVEL
STORAGE
LAKE STORAGE LEVEL
MINIMUM DIAMETER DRAIN-DOWN PIPE
ATTEMPT TO SET STORAGE LEVEL AT OR ABOVE RECORDED LOW WATER TABLE READING
30 MIN
6' MIN.
STABILIZED EARTHEN BERM
TYPICAL 1' PONDING
GRASSED SWALE
HOUSE PAD AT 100 yr FLOOD ELEV MIN
1' MIN
6" MIN
GRASSED ROADSIDE SWALES
Ⓐ
STORM WATER CONVEYANCE SYSTEM GRASSED SWALES
60% OF LOT DRAINS TO REAR YARD SWALE
40% OF LOT DRAINS TO STREET SWALE
MINIMUM STREET ELEVATION 1' ABOVE DESIGN HIGH WATER
TO DOWNSTREAM CONVEYANCE NETWORK
MINIMUM 1' ABOVE DESIGN HIGH WATER
DESIGN HIGH WATER
CONTROL STRUCTURE
LAKE CONTROL LEVEL
STORAGE
LAKE STORAGE LEVEL
MINIMUM DIAMETER DRAIN-DOWN PIPE
ATTEMPT TO SET STORAGE LEVEL AT OR ABOVE RECORDED LOW WATER TABLE READING
6' MIN
GRASSED SWALE
HOUSE PAD AT 100 yr FLOOD ELEV MIN
GRASSED ROADSIDE SWALES
Ⓑ
STORM WATER CONVEYANCE SYSTEM GRASSED SWALES/STORM SEWERS
NOTE: DRAWING NOT TO SCALE

vegetation (on Deltona property) before entering open receiving waters.

The major drainage control structures are designed to maximize storage and detention while both assuring flood protection to the upland properties and allowing for positive drainage (by means of small diameter pipes) to regain storage potential after storm events.

The structure design typically prevents the lowering of ground water levels below historic low readings. The structures themselves are to function as weirs but their ultimate shape and characteristics (construction material) can vary so long as they provide the discharge characteristics the design was based on. The structural designer may find it aesthetically pleasing and cost effective to construct stabilized and protected earthen weirs or may elect to adhere to a more conventional solution by erecting concrete structures. Such decisions will be made at the time of platting and submittal of detailed drainage plans.

The grassed swales in street right-of-ways, rear yards and conveyance networks are to be shallow (typically 6" deep) with minimum slopes (identical to street slopes) to maximize contact time and facilitate particulate removal and nutrient scrubbing. The placement of water and sewer (force) mains within the swale areas along the streets will effectively penetrate any "hard-pan" soil layer that might exist, thereby promoting additional infiltration and percolation.

Lake areas will be constructed with gentle side slopes (5:1) extending to a minimum of 3 feet below storage levels (see Figure 1) to prevent bank sloughing and to increase littoral zone so as to enhance nutrient removal efficiencies. All lake areas shall be deeper than 6 feet as measured from storage level to prevent the establishment of cattails and other rooted aquatics in open water areas.

Procedures

The analysis of the drainage system was performed using the Soil Conservation Service's (SCS) computer program TR-20. The TR-20 program is an event model capable of generating basin runoff hydrographs, adding hydrographs and routing hydrographs through reservoirs and reaches. All computer work was performed on a CHI 2020 with graphic output generated on a Calcomp 633 Roll Plotter.

The TR-20 employs the unit hydrograph method to generate design stormwater runoff characteristics. Output from the program includes time and quantity at peak discharge and volume of runoff on the basin.

Input data requirements include the designation of both a dimensionless hydrograph representative of the study area, and a dimensionless rainfall distribution curve. Due to the characteristics of the property (gentle topographic relief, large wetland slough system) a dimensionless hydrograph with a peak rate factor of 325 was selected.

The selected rainfall pattern is described by the SCS as being a "B" type distribution appropriate for use with design storms of 24 hour duration. Additional input requirements include the designation of the design storm, antecedent soil moisture condition, time of concentration of the basin, area of basin, and composite Complex Number (CN) value. Design storm event quantities were taken from Technical Paper-40 (TP-40).

Antecedent soil moisture was assumed to be 2 (average) under design conditions but was raised to 3 (wet) when the system was loaded with a 2-year - 24 hour event at a time when all system storage was depleted. The time of concentration, i.e., time it takes for a drop of water furthest away from the basin outfall to reach the outfall, was computed using assumed velocities for each type of travel segment. The area of each basin was determined by planimeter. The composite basin CN value was computed by proportionately weighing the CN values assigned to the various basin land uses or was taken from TR-55. Soil mapping of the property was performed by the Soil Conservation Service.

Soil hydrologic ratings were also taken from Technical Paper-55 (TR-55) and for the developed condition the well drained rating for each soil type was employed.

The TR-20 program will route inflow hydrographs through a reservoir if a stage-storage-discharge relationship for the reservoir can be described. Output from the procedure includes a stage-discharge (vs. time) hydrograph and the designation of the time and quantity at peak discharge as well as the volume of runoff on the basin.

Discharge data from the TR-20 reservoir routing procedure can be incorporated into the HYDRA program, developed by the author, which generates a graphic output on the Calcomp Plotter.

The author has also developed a computer program named TR-20D, which, when given basin and drainage control structure characteristics, will describe the stage-discharge-stage relationship for the area in a format suitable for introduction into the TR-20. In general, only lake storage was considered in the design condition. However, a storage credit of 3-inches was applied to each contiguous drainage retention area (DRA) greater than 50 acres in size, and 1-inch of runoff storage was assigned to all commercial, industrial and multi-family areas. (to be enforced by deed restriction)

An iterative approach employing the TR-20 program, known as the

Conservative Weir Analysis (CWA), was developed by the author for use in the design of the subject drainage works. The benefits enjoyed by employing the CWA approach include:

1. Compatibility with TR-20 which generates discharge hydrographs and elevation profiles.
2. Conservative solutions since it assumes free flow hydraulic conditions (over weirs and through pipes) but is designed to allow headwater, tailwater equalizations (with pipes) yielding lower actual flows.
3. Conservative solution since the peak elevation resulting from the 25 year - 24 hour design storm will be well below the lowest street in each watershed, and for less frequent or multi-event storms one can allow an additional rise in head (above design highwater) of up to 6" over the storage area (where flow over the weir is increasing at $h^{(3/2)}$) before the upstream weir becomes submerged.
4. Simple and accurate discharge relationship can be defined for a weir.
5. Provides for maximum storage and detention yielding water at a lower rate of flow and higher quality:
    a. Promotes settlement of particulate matter.
    b. Increases recharge capacity of the system.
    c. Provides for nutrient absorption.
    d. Prevents "shock" loadings to receiving water bodies.
6. Computer solution allows for the evaluationof numerous alternatives quickly in order to achieve a "best" overall design at a relatively inexpensive cost to user.

The CWA approach requires that each major drainage control structure be a weir with minimum pipe(s) (18-inch) placed through it. Major conveyance structures within each basin were assumed to be sized so as to prevent significant increases (greater than 0.2') in water surface profile through them. Secondary drainage works are assumed to be sized so as to prevent local flooding for the 10 year - 24 hour event.

The CWA approach requires that each structure be capable of conveying the maximum flow for the design storm over the weir alone (no pipe discharge contributions) and at no time shall water elevations exceed design high water. Simply stated the TR-20 analysis is run twice for each structure, once with no pipe in the weir to assure that water elevation will not rise above design level, and again with a pipe to compute total possible flow to downstream structure.

The steps involved in the CWA design approach are as follows:

1. Select pipe invert elevation, weir elevation and length, and design high water, specify basin storage characteristics and input into TR-20D. Output from TR-200 includes stage-discharge-storage relationships for the structure with and without pipe(s).
2. Insert structure data (without pipe) generated in step 1 into TR-20.
3. Using TR-20 generate design inflow runoff hydrograph and route through structure described in step 2.
4. Evaluate results from step 3 and determine whether peak elevation exceeds design high water. If peak elevation is greater than design high water go back to step 1 and redesign. Likewise, if peak elevation is significantly lower than design consider starting again at step 1. If acceptable, continue.
5. Insert structure data (with pipe) generated in step 1 into TR-20.
6. Go to step 1 and design downstream structure, then repeat procedures

2 through 6 until all downstream structures have been designed.

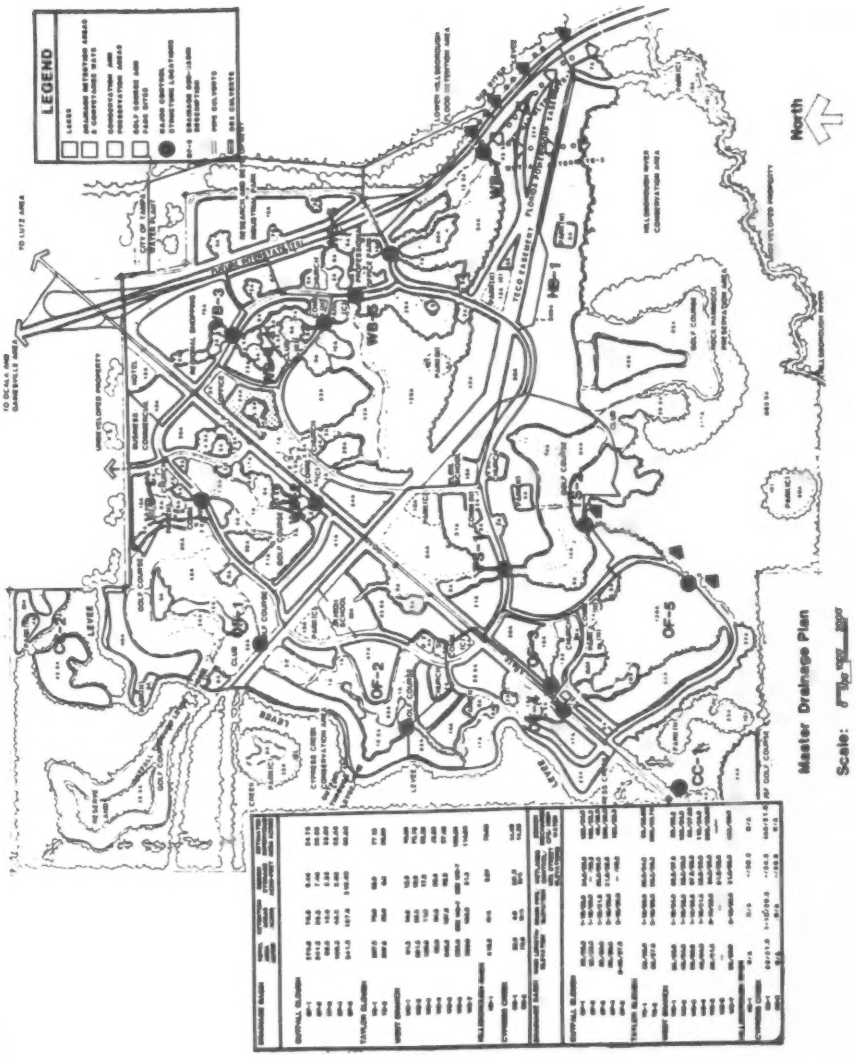
Once the design of each basin is completed the evaluation of other storm events is easily accomplished by use of TR-20. In this study storms of 50, 10, 5 and 2 year frequency and 24 hour durations were also investigated assuming design storage was available and soil moisture conditions were average (2). In addition, a 2 year - 24 hour storm was applied to the system for a wet soil condition (3) with no storage available. It is also of interest to note that we have investigated the 25 year - 5 day storm (14.5 inches distributed on a "B"-type SCS rain curve) and have found that peak discharges fall within, and typically 10% below those resulting from the 25 year - 24 hour event, thereby supporting the selection of critical design storm duration.

Basin Description

In its developed condition, Tampa Palms will be composed of 6 drainage basins as illustrated on Figure 2. All areas of the property will maintain their historic hydrologic relationship and patterns of flow except for those areas north of SR-581, east of the proposed Cypress Creek Levee which have historically drained to Cypress Creek. However, steps have been taken to prevent hydrologic impacts to Cypress Creek by proposing the incorporation of pipe structures through the levee at several historic outfall points. These structures will allow the maintained upland flows to Cypress Creek during low stage period of the Creek. In addition, the maintenance of relative historic ground water levels (by staging storage areas) within the areas adjacent to Cypress Creek will allow for continuance of the ground water contribution from the property.

Two of the developed drainage basins (HB-1 and CC-2) require no major

Figure 2. Master Drainage Plan


LEGEND
Master Drainage Plan
Scale:
North
WB-1
WB-3
OF-2
OF-5
CC-1
GOLF COURSE
LEVEE
CLUB
HOTEL
TO OCALA AND GAINESVILLE AREA
TO LUTZ AREA

drainage works or internal retention areas as a result of their limited breadth and setting in the project. The four remaining basins ranged in size from a minimum of 28.0 acres for CC-1 to a maximum of 1660.4 acres for the West Branch drainage basin (WB-1 through WB-7). In the interest of space only results for the West Branch basin are reported herein.

Only limited areas of property not owned by Deltona contribute to the proposed drainage system. It is of importance to note that the land uses of the offsite property are well defined and not subject to significant changes, which might impact the design of the drainage system. Offsite areas contributing to the drainage system are classified as: roads (SR 581, I-75 extension); power easements (TECO and FPC); utility sites (City of Tampa Water Plant); and open space adjacent to S.W.F.W.M.D. Levee-112 N.

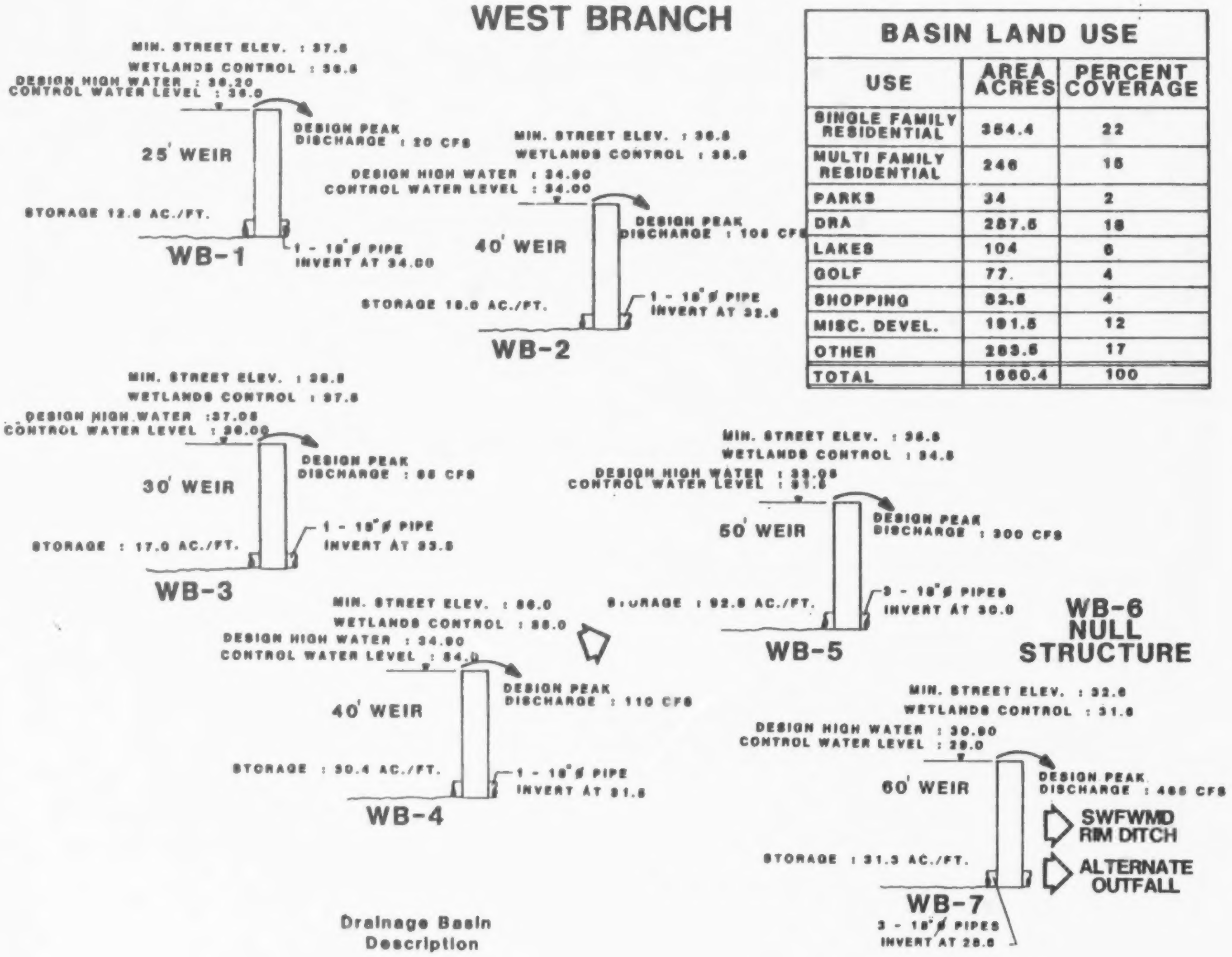
Lands north of the Deltona property boundary may have historically drained through portions of Deltona property to Cypress Creek. Drainage from that small area of offsite lands that could be deprived of their natural outfall as a result of Cypress Creek Levee construction, will be conveyed (via shallow swales on Deltona property) to low lying wetland areas within Deltona property outside of the levee. This will prevent unnatural flooding of the offsite property and relieve Deltona's "controlled" (development restrictions and operation) drainage system from unquantifiable externalities that would otherwise be imposed upon it.

Results for the West Branch Basin

The developed West Branch drainage basin contains 1660.4 acres and is composed of 7 sub-basins, the composite areas of which are included in its existing undeveloped drainage basin. A breakdown of the basin land uses and a description of the control structures and design storm results are included on Figure 3. Historically these areas drained to Trout Creek; however, with the implementation of the Corps of Engineers Lower Hillsborough Flood Detention Area project this natural outfall will be eliminated (by Levee 112N construction) and future flows will be directed to the Hillsborough River via a conveyance ditch. It will be necessary for Deltona to provide for additional outfall capacity for this basin since the design of the present Levee conveyance ditch will only be capable of handling a portion of the projected discharge from the design event. Graphic results of the analysis of the design storm at the outfall structures (25 years - 24 hour event) are included on Figure 4. This figure depicts: estimated pre-development discharge vs. time; simulated post development discharge vs. time assuming no control structures (inflow); design post development discharge vs. time with control structures (out flow); and cumulative percent discharge vs. time for the post-development design storm with control structures. Design development control discharges are considered to be within reasonable accord when compared with pre-developed conditions in regard to both total quantities and peak flows. Any discrepancies can be explained as resulting from reinforcing conservative analysis approaches for pre and post-development studies.

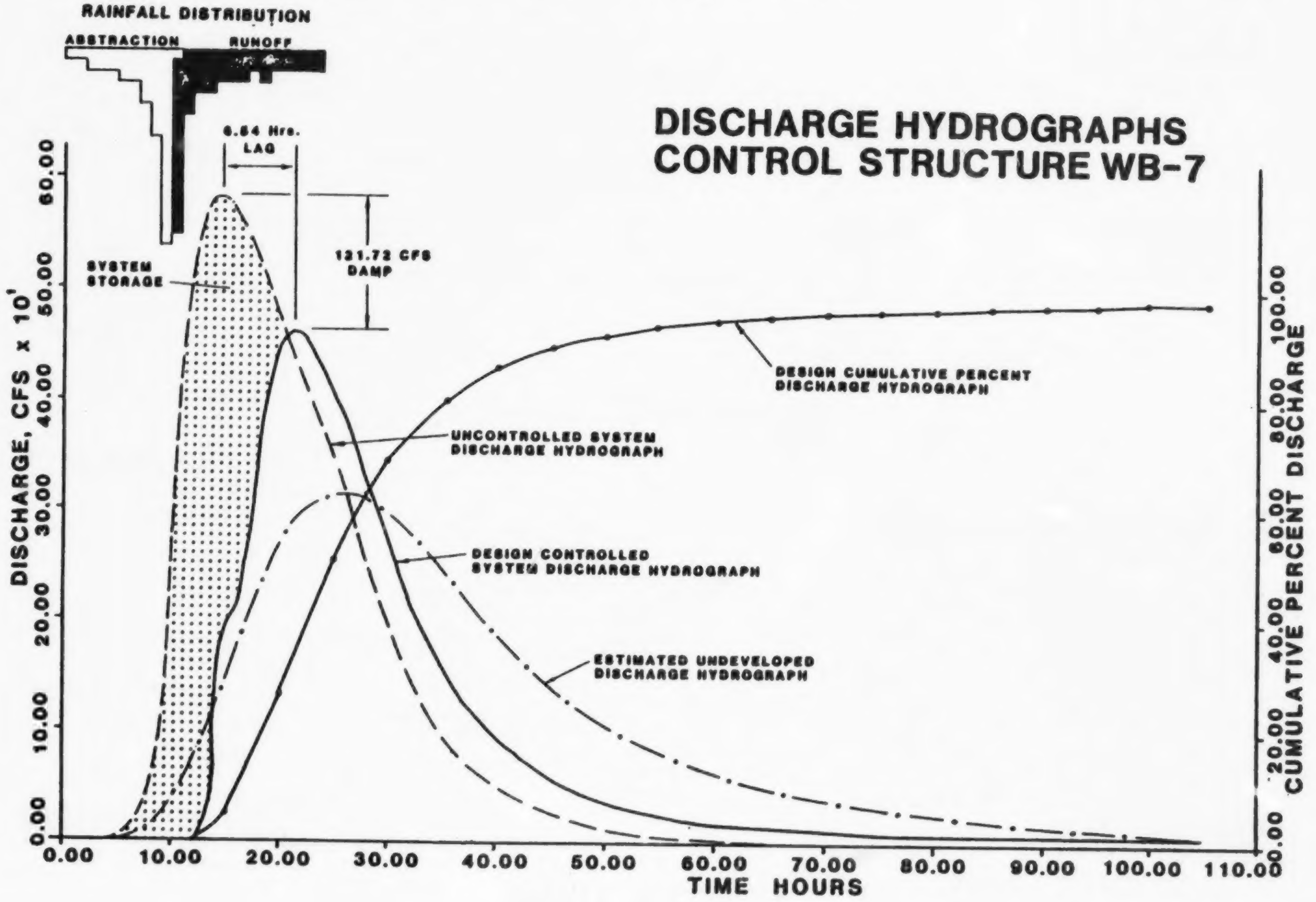
Review of the curve illustrating the percentage of cumulative discharge reveals that only 26.8% of the total discharge quantity is released after the first 20 hours of the storm event. In the same regard, 52.0%, 81.4% and 91.5% of the total discharge volumes are released after 25, 35, and 45 hours, respectively, into the storm event. These values indicate desirable retention capabilities which will provide for increased storm water quality enhancement and recharge potential.

Figure 3. West Branch Drainage Characteristics


WEST BRANCH
MIN. STREET ELEV. : 37.5
WETLANDS CONTROL : 36.5
DESIGN HIGH WATER : 36.20
CONTROL WATER LEVEL : 36.0
25' WEIR
DESIGN PEAK DISCHARGE : 20 CFS
STORAGE 12.6 AC./FT.
1 - 18" ø PIPE INVERT AT 34.00
WB-1
MIN. STREET ELEV. : 36.5
WETLANDS CONTROL : 35.5
DESIGN HIGH WATER : 34.90
CONTROL WATER LEVEL : 34.00
40' WEIR
DESIGN PEAK DISCHARGE : 105 CFS
STORAGE 16.0 AC./FT.
1 - 18" ø PIPE INVERT AT 32.6
WB-2
MIN. STREET ELEV. : 38.5
WETLANDS CONTROL : 37.5
DESIGN HIGH WATER : 37.05
CONTROL WATER LEVEL : 36.00
30' WEIR
DESIGN PEAK DISCHARGE : 65 CFS
STORAGE : 17.0 AC./FT.
1 - 18" ø PIPE INVERT AT 33.5
WB-3
MIN. STREET ELEV. : 36.0
WETLANDS CONTROL : 35.0
DESIGN HIGH WATER : 34.90
CONTROL WATER LEVEL : 34.0
40' WEIR
DESIGN PEAK DISCHARGE : 110 CFS
STORAGE : 30.4 AC./FT.
1 - 18" ø PIPE INVERT AT 31.5
WB-4
MIN. STREET ELEV. : 36.5
WETLANDS CONTROL : 34.5
DESIGN HIGH WATER : 33.05
CONTROL WATER LEVEL : 31.5
50' WEIR
DESIGN PEAK DISCHARGE : 300 CFS
STORAGE : 92.8 AC./FT.
3 - 18" ø PIPES INVERT AT 30.0
WB-5
WB-6 NULL STRUCTURE
MIN. STREET ELEV. : 32.6
WETLANDS CONTROL : 31.6
DESIGN HIGH WATER : 30.90
CONTROL WATER LEVEL : 29.0
60' WEIR
DESIGN PEAK DISCHARGE : 465 CFS
SWFWMD RIM DITCH
ALTERNATE OUTFALL
STORAGE : 31.3 AC./FT.
WB-7
3 - 18" ø PIPES INVERT AT 28.6
Drainage Basin Description

BASIN LAND USE		
USE	AREA ACRES	PERCENT COVERAGE
SINGLE FAMILY RESIDENTIAL	354.4	22
MULTI FAMILY RESIDENTIAL	246	15
PARKS	34	2
DRA	287.5	18
LAKES	104	6
GOLF	77.	4
SHOPPING	82.5	4
MISC. DEVEL.	191.5	12
OTHER	283.5	17
TOTAL	1660.4	100

Figure 4
West Branch Discharge Hydrograph



Results of hydraulic analysis for each sub-basin for various storm events are presented on Table 1. It is of interest to note that even under severely adverse drainage conditions (storage completely depleted, soil moisture condition = 3) the introduction of a 2 year 24 hour storm (5.5 inches) does not cause increases in discharges of greater than 10% for selected basins and in no cases would these increased levels impose adverse impacts to the receiving areas, development or drainage network. With a total 203.1 acre-feet of storage available in design conditions, this basin can retain (conservatively estimated) 1.47 inches of runoff, 3.70 inches of rainfall), before ultimate discharge over the outfall weir.

In conjunction with additional DRA storage, heretofore not considered in design, the system will experience reductions in peak flows under design conditions with an improved ability to manage storm water qualities. It should also be noted that the practices of draining 60% of the lots abutting open space towards the open space, and requiring on-site retention for the first inch of runoff on all multi-family, commercial and industrial areas will further reduce the direct hydraulic effectiveness of impervious areas thereby reducing flows and pollutant loads.

Quantitative estimates have been made of the time it would take to lower storage levels within the system from the weir control levels to the normal lake storage levels (design storage level). Excluding evapotranspiration and seepage losses, levels are expected to return to normal within 30 days. Sub-basin WB-5 is the governing area in this regard in comparison to up-gradient basins which will return to normal within 10 to 14 days.

Table 1
West Branch Drainage Analysis Summary

WEST BRANCH

Structure	Storm Event Amount of Rain Soil Condition	25 Yr.-5 day 14.5" (2)	25 Yr.-24 Hr. 9.5" (2)	10 Yr.-24 Hr. 8.5" (2)	5 Yr.-24 Hr. 7.5" (2)	2 Yr.-24 Hr. 5.5" (2)	2 Yr.-24 Hr. 5.5" Lakes Full (3)
WB-1	Pk. Disch. (Time)	20.20 (53.55)	18.97 (18.06)	12.54 (21.40)	8.78 (23.47)	5.42 (24.39)	20.35 (13.41)
	Pk. Elev.	36.22	36.20	36.05	35.84	35.32	36.43
	In. On Basin	10.40	5.89	4.99	4.15	2.52	3.86
WB-2	Pk. Disch. (Time)	101.20 (53.36)	103.67 (17.30)	84.87 (17.00)	67.81 (17.64)	36.09 (21.05)	82.60 (15.14)
	Pk. Elev.	34.86	34.88	34.75	34.64	34.40	34.81
	In. On Basin	10.16	5.56	4.69	3.85	2.24	3.06
WB-3	Pk. Disch. (Time)	55.29 (49.46)	85.10 (12.58)	71.82 (12.85)	59.09 (13.13)	30.73 (14.5)	62.48 (11.58)
	Pk. Elev.	36.72	37.04	36.90	36.76	36.44	36.82
	In. On Basin	11.44	6.66	5.72	4.83	3.03	4.43
WB-4	Pk. Disch. (Time)	94.19 (50.63)	108.24 (15.31)	79.48 (16.36)	60.19 (17.42)	22.13 (23.36)	92.34 (12.80)
	Pk. Elev.	34.84	34.91	34.74	34.56	34.21	34.87
	In. On Basin	10.90	6.13	5.22	4.32	2.54	4.40
WB-5	Pk. Disch. (Time)	320.30 (55.03)	298.58 (21.26)	238.55 (22.42)	182.73 (24.20)	75.06 (29.72)	237.13 (17.49)
	Pk. Elev.	33.14	33.06	32.83	32.62	32.10	32.87
	In. On Basin	9.64	4.93	4.05	3.18	1.53	4.00
WB-6				Null Structure			
WB-7	Pk. Disch. (Time)	501.04 (54.97)	463.60 (21.24)	372.58 (22.37)	286.33 (24.13)	116.97 (29.32)	365.61 (17.69)
	Pk. Elev.	31.00	29.88	29.59	29.31	28.76	29.55
	In. On Basin	3.82 (67.7%)	5.15 (54.21%)	4.27 (50.24%)	3.39 (45.2%)	1.73 (31.45%)	4.02 (73.10%)

Notes:

Pk. Disch.= Peak discharge, values in cubic feet per second
Time= Time of peak discharge from beginning of rain event
Pk. Elev.= Peak elevation, values in feet, NGVD
In. On Basin= Amount of runoff on basin in inches.

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WATER QUALITY IN THE FIRST SECOND AND THIRD ORDER STREAMS OF AN UPLAND AND FORESTED WETLAND WATERSHED

by

Charles M. Courtney[1]

INTRODUCTION

As part of a multidisciplinary approach to the private development of the 2200 hectare Tampa Palms, Florida, land use and the ambient background water quality of adjacent streams are described in this paper. The Tampa Palms site (Figures 1 and 2) is located between the confluence of two, intermittant, second order streams and the Hillsborough River, which is a potable water source for the city of Tampa. As is typical of central west coastal Florida there is gentle relief to the terrain which slopes from 44.0 feet to 24.5 feet from north to south across the property. DuBois and Courtney (1979) have described land use on the site according to the Florida Land Use and Cover Classification System (Anon., 1976). This system not only divides the land among several broad categories (i.e. agricultural, rangeland, forested uplands, and wetlands) but it also subdivides these categories based on specific uses or dominant species asso-

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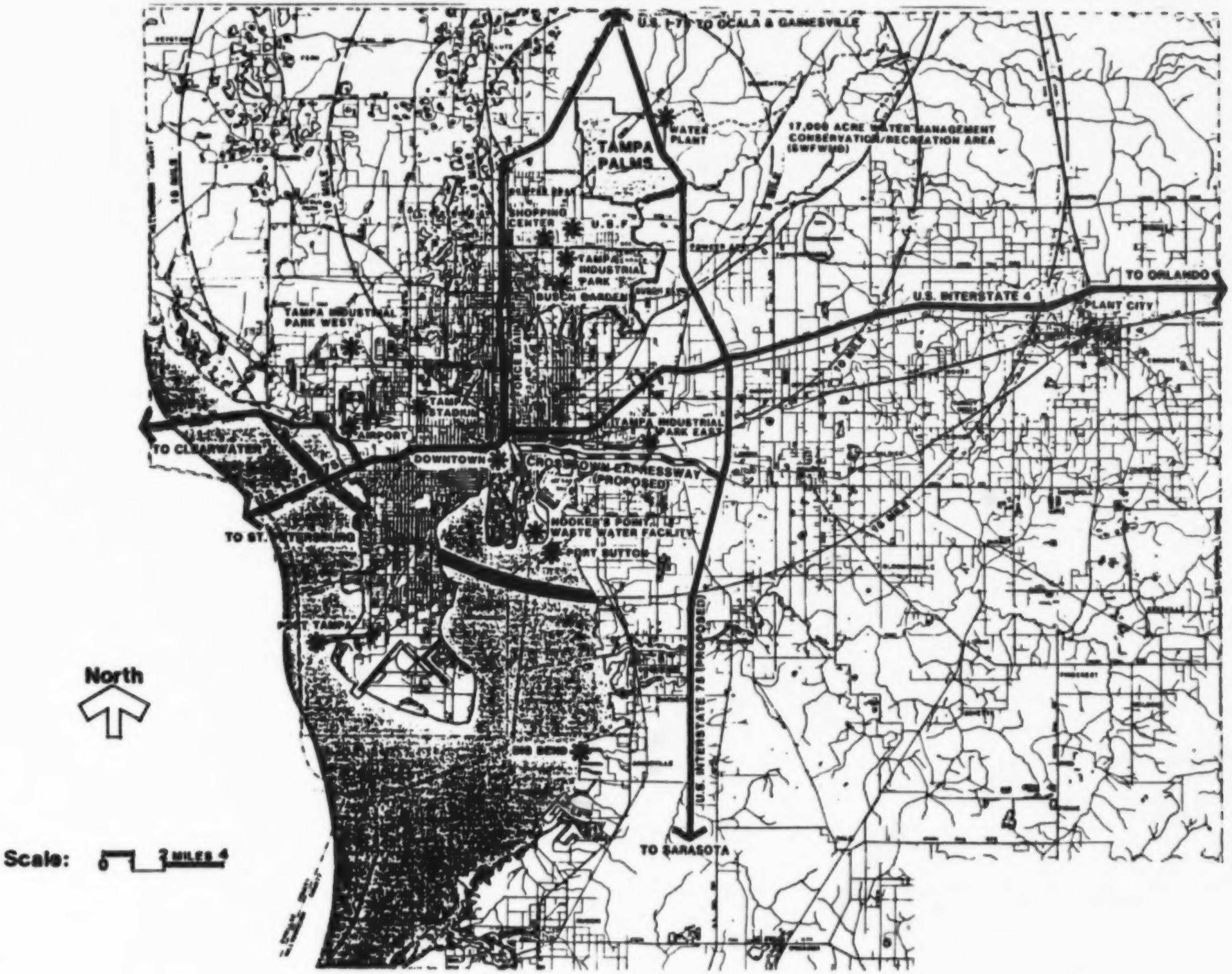
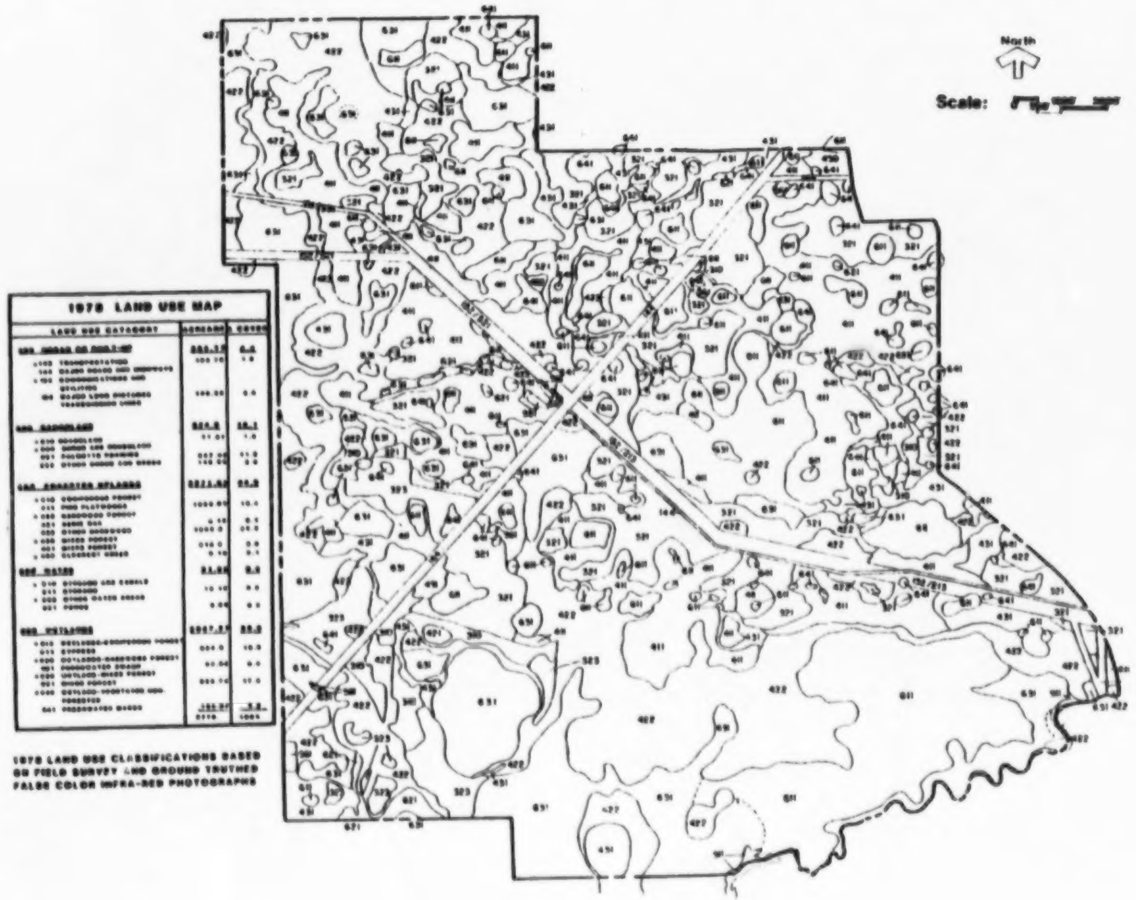
Figure 1. A Map Showing the Location of the Tampa Palms Study Site


U.S. I-75 TO OCALA & GAINESVILLE
WATER PLANT
TAMPA PALMS
17,000 ACRE WATER MANAGEMENT CONSERVATION/RECREATION AREA (SWFWMD)
SHOPPING CENTER
U.S.F.
TAMPA INDUSTRIAL PARK
BUSCH GARDENS
TAMPA INDUSTRIAL PARK WEST
TAMPA STADIUM
AIRPORT
TO CLEARWATER
DOWNTOWN
CROSSTOWN EXPRESSWAY (PROPOSED)
TAMPA INDUSTRIAL PARK EAST
HOOKER'S POINT WASTE WATER FACILITY
PORT SUTTON
TO ST. PETERSBURG
U.S. INTERSTATE 75 (PROPOSED)
TO SARASOTA
BIG BEND
U.S. INTERSTATE 4
TO ORLANDO
PLANT CITY
North
Scale: 0 2 MILES 4

Reproduced from best available copy.

Figure 2
A Land Use Description* for the Tampa Palms Development Site


1978 LAND USE MAP
1978 LAND USE CLASSIFICATIONS BASED ON FIELD SURVEY AND GROUND TRUTHED FALSE COLOR INFRA-RED PHOTOGRAPHS
North
Scale:

*DuBois and Courtney (1979)

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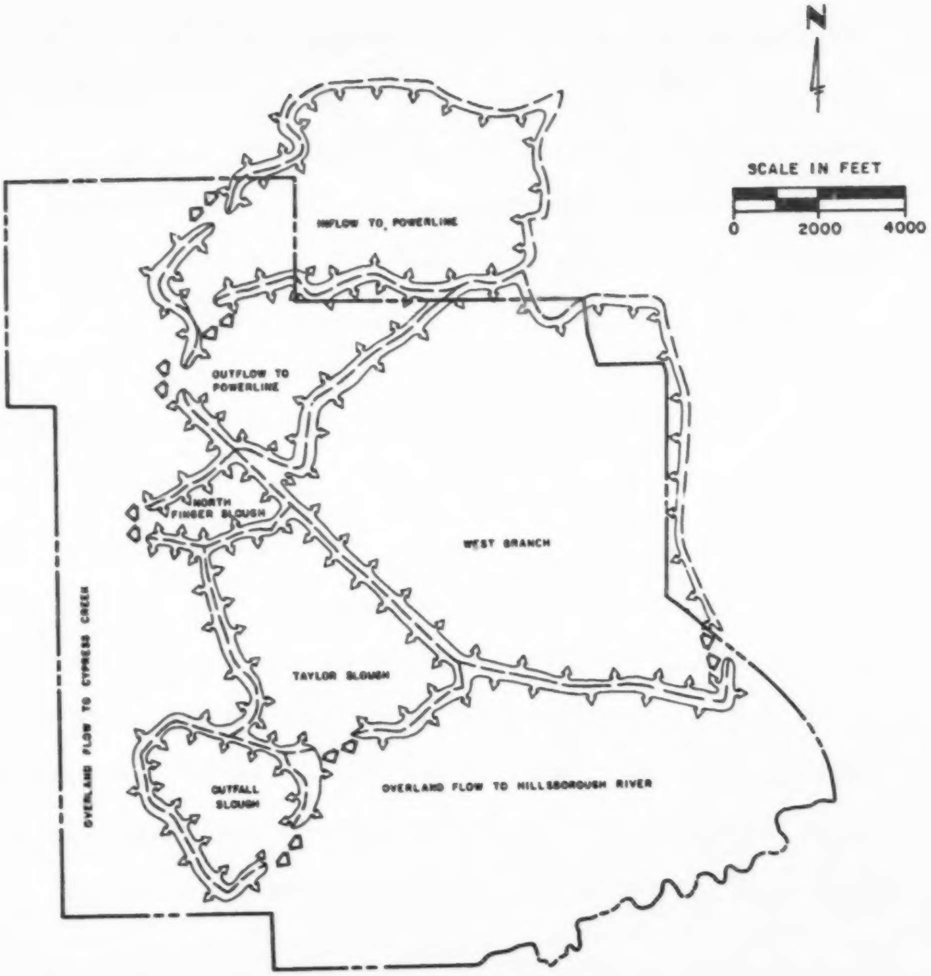
ciations (e.g. pine flatwoods, xeric oak hardwoods and cypress forested wetlands). The forested uplands of the site (Figure 2) were shown to comprise 44.6% of the land space, and they were dominated by Quercus laurifolia with other hardwoods (22.2%); (slash) pine flatwoods (18.4%); some mixed forest (3.8%) where live oak and slash pine were intermixed; xeric oak (Quercus geminata) (.1%); and clearcut areas (.1%). Additional uplands were in use as rangeland (15.1% of land space) and these were dominated by palmetto prairies (11.2%) with minor components (2.6%) of other shrub or brush species and grasslands (1.3%). Little of the land was in urban use (4.4%) and that consisted of a powerline right of way (2.6%) and the space occupied by State Road (SR) 581 (1.8%). The wetlands that drain these uplands occupied another 35.5% of the land space. A mixture of Fraxinus, Quercus, Acer and Taxodium dominated communities made up 17.3% of the wetlands; cypress dominated forests made up 15.5%; while freshwater marsh made up 2.3% and freshwater swamps only made up 0.4%.

Motchkavitz (1979) defined six major internal drainage basins which discharged from the property at well defined points (Figure 3) with the remainder of the property outside of these areas draining overland to either Cypress Creek or to the Hillsborough River.

Figure 3
A Map* Showing the Location of the Six Pre-Development Basins at Tampa Palms


N
SCALE IN FEET
0
2000
4000
INFLOW TO POWERLINE
OUTFLOW TO POWERLINE
NORTH FINGER SLOUGH
WEST BRANCH
TAYLOR SLOUGH
OUTFALL SLOUGH
OVERLAND FLOW TO CYPRESS CREEK
OVERLAND FLOW TO HILLSBOROUGH RIVER

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*Motchkavitz (1979)

The water quality of Cypress Creek was described by the Southwest Florida Water Management District (Anon., 1975) and Briley Wild Associates, Inc. (Anon., 1978). This second order stream and another, Trout Creek, flow south from Pasco County, Florida to the Hillsborough River but resemble riverine swamps adjacent to the property. The property is separated from Trout Creek by a dike that SWFWMD is using to retain Hillsborough River floodwaters and thereby protect the city of Tampa.

The Hillsborough River is a first order stream which originates in central Florida and flows westward through rich phosphate deposits (Jones et al., 1973) to Hillsborough Bay.

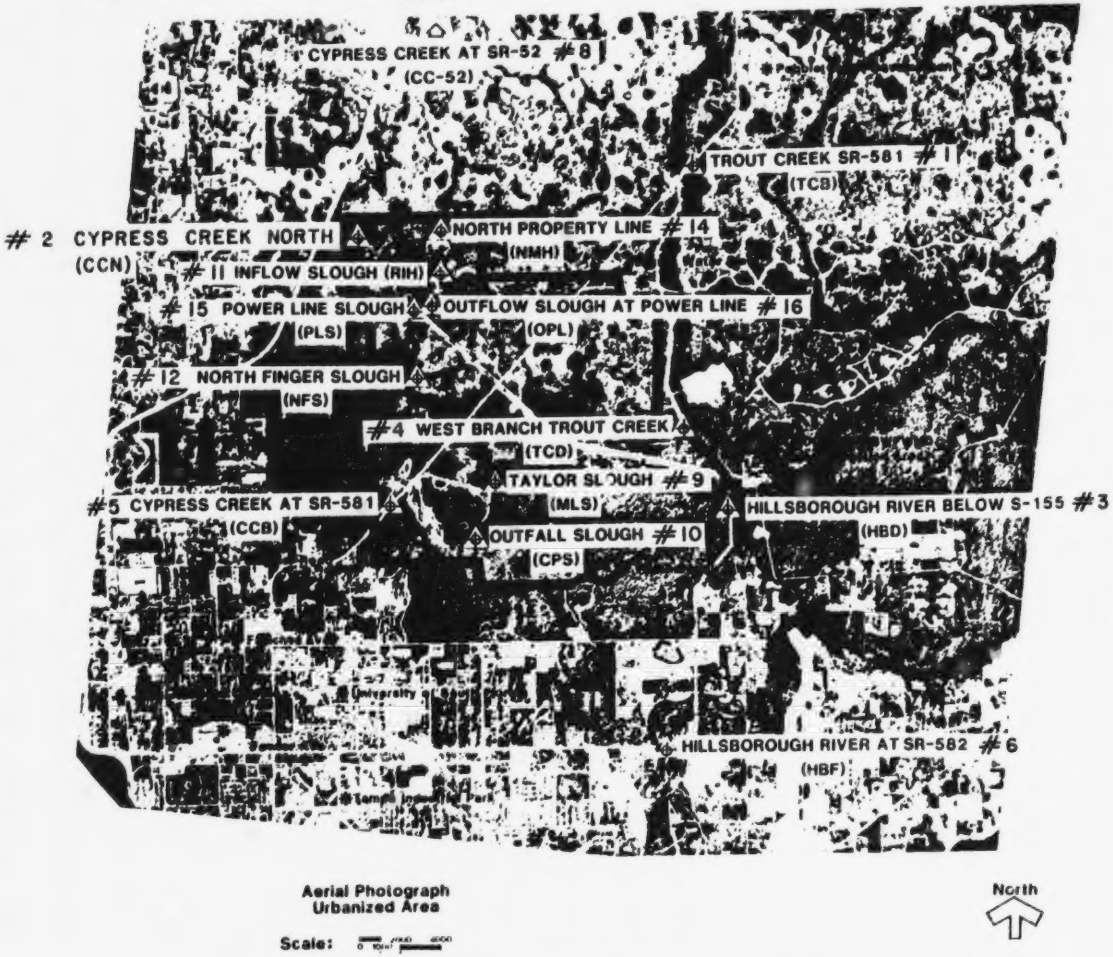
METHODS AND MATERIALS

Ambient receiving water chemical and microbiological data were collected at regular monthly intervals at five stations (Figure 4) over the period October 1978 to December 1979.

Station 1 was located at the intersection of Trout Creek and SR 581 and described the water quality of Trout Creek upstream of the property while station 4 was located at the mouth of the West Branch of Trout Creek (an intermittant tertiary stream) where it discharged from the property. Station 2 was in Cypress Creek at the northwest corner of the site and represented the water quality of this creek as it entered the property while station 5 at the intersection of the creek and SR 581 was representative of water quality in the creek as it exited the property. Station 3 was located in the Hillsborough River upstream of the property. Monthly collections were also made at station 6 in the Hillsborough River over the period February - December 1979 to represent water in the River downstream of the property.

Since the discharge points for the internal drainage basins of the property were well defined, a second series of stations were designated to monitor the quality of runoff from selected storm events. Each storm event sample series con-

Figure 4. A Map Showing the Water Quality Monitoring Sites


CYPRESS CREEK AT SR-52 #8
(CC-52)
TROUT CREEK SR-581 #1
(TCB)
# 2 CYPRESS CREEK NORTH
(CCN)
NORTH PROPERTY LINE #14
(NMH)
#11 INFLOW SLOUGH (RIH)
#15 POWER LINE SLOUGH
(PLS)
OUTFLOW SLOUGH AT POWER LINE #16
(OPL)
#12 NORTH FINGER SLOUGH
(NFS)
#4 WEST BRANCH TROUT CREEK
(TCD)
TAYLOR SLOUGH #9
(MLS)
#5 CYPRESS CREEK AT SR-581
(CCB)
HILLSBOROUGH RIVER BELOW S-155 #3
(HBD)
OUTFALL SLOUGH #10
(CPS)
University
HILLSBOROUGH RIVER AT SR-582 #6
(HBF)
Aerial Photograph
Urbanized Area
Scale:
North

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sisted of samples which were collected symetrically on the discharge hydrograph. Storm runoff was differentiated by a period of no flow at the station for seven days prior to the storm event and the occurrence of no rainfall on the property during that period.

Water quality at station 4 represented storm runoff from the West Branch basin which was the largest internal basin on the property (609 hectares). This basin contains 41% forested uplands, 32% wetlands and 27% rangelands and contributes ∿ 10% of Trout Creek's overall flow (Motchkavitz, 1979). Station 9 water quality represents storm runoff from the (195 hectare) Taylor Slough basin which is evenly divided between rangeland (39%), wetland (33.7%) and forested uplands (27.3%). Water quality at station 10 represents storm runoff from the Outfall Slough basin which consists of 56.8% wetlands, 33.1% rangeland, and 10.1% forested uplands. The North Finger Slough Basin is the smallest basin on the property (42 hectares) and is dominated by forested uplands (63.6%) with small but equal portions of rangeland and wetlands (18.6% and 17.8%, respectively). Water samples from station 12 represent storm runoff from this basin. The Outflow Slough basin in the northwest corner of the property is dominated by wetlands (45.9%), with 34.4% forested uplands and 19.7% rangeland. Station 15 water collections represent storm runoff from this basin, but it should be noted that the boundaries of this basin are questionable because some flow has been observed entering its northwest corner from the Inflow Slough Basin. Because some of the regular monitoring stations in second order streams (e.g. Cypress Creek) were also intermittant some storm event sampling also took place at these locations during runoff pulses.

For regular monthly monitoring and storm event stations *in situ* measurements of temperature, conductivity and dissolved oxygen were made using a Hydrolab System 2000 while pH was determined in the field using an Orion Model 401 Specific Ion meter. Turbidity was determined by the Nephelometric Method (APHA, 1976) on a Hach Model 1860A laboratory turbidimeter. Total filtrable residue, nonfiltrable residue, BOD_5, Color, ammonia-nitrogen, total phosphate, chlorophyll *a*, phaeophytin *a*, and total, fecal and fecal streptococcal bacteria were determined by the methods of APHA (1976). Nitrate-nitrogen, nitrite-nitrogen, and total organic carbon were measured by the methods of EPA (1974) while total kjeldahl nitrogen was analyzed by the soluable organic nitrogen digestion of Strickland and Parsons (1972). Data from background *and* storm event sampling were combined for each station for between station tests of significant difference for the respective parameters using the BMDP 3D program of Dixon and Brown (1979).

RESULTS

A summary of the chemical and physical characteristics of background and storm event monitoring at Tampa Palms is shown in Table 1. Four storm events were analyzed and they included 1.2, 1.4, 2.8 and 10.5 inch events. Total fecal and fecal strep counts were always highest in Third Order streams and lowest in Second Order streams. Counts in the Hillsborough River averaged 33% lower at the upstream station 3. Combined nitrate and nitrite nitrogen levels decreased from first to second to third order streams. Ammonia nitrogen was not measured in the tertiary streams, but it was higher in second order streams than in the Hillsborough River. Total kjeldahl nitrogen levels were highest at downstream stations in first and second order streams and lowest in third order streams. Total phosphate was highest in the Hillsborough River as was reactive phosphate. Highest average BOD_5 levels were found in the third order streams while TOC concentrations were highest in second order streams. The water was also generally more colored in second order streams than in the Hillsborough River.

TABLE 1 A SUMMARY OF BACKGROUND AND STORM RUNOFF WATER QUALITY IN FIRST, SECOND AND THIRD ORDER STREAMS

		COLIFORM ($*x10^4$/100 ml)												SOLIDS*						
		TOTAL	FECAL	F. STREP	NO_2N+ NO_3N*	NH_3N*	TKN*	TPO_4*	OPO_4*	BOD_5*	TOC*	CHL*a***	PHAE*a***	T.DIS.	SUSP.	COLOR (Apt-Co)	TURB. (NTU)	pH	TEMP. (°C)	COND. (μmhos/cm)
FIRST ORDER STREAM-MONTHLY BACKGROUND SAMPLING + STORM EVENT SAMPLING																				
STATION NO.																				
3-HILLSBOROUGH	n	9	10	10	16	16	17	17	16	13	12	10	10	14		14	14	6	5	4
RIVER	$\bar{x}$	1.06	.02	.02	.426	.024	.63	.375	.358	1.3	12.6	8.6	2.3	182		107	2.5	7.42	19.7	266
6-HILLSBOROUGH	n	8	8	8	11	11	11	11	11	11	8	10	10	10		11	11	5	6	5
RIVER B	$\bar{x}$	.11	.04	.35	.217	.025	1.20	.383	.369	1.0	14.1	3.0	3.7	167		138	2.2	7.1	18.4	232
6-HILLSBOROUGH	n						3	3		1	3									
RIVER S	$\bar{x}$						1.35	.308		.2	15									
SECOND ORDER STREAM-MONTHLY BACKGROUND SAMPLING + STORM EVENT SAMPLING																				
STATION NO.																				
2-CYPRESS	n	9	9	9	16	16	16	16	16	13	10	10	10	14		14	14.	8	7	6
CREEK B	$\bar{x}$	.06	.005	.02	.118	.037	.79	.047	.028	1.4	25.4	2.6	2.1	164		193	1.7	6.91	17.7	192
2-CYPRESS	n						3	3		1	3									
CREEK S	$\bar{x}$						.77	.019		.7	20.3									
5-CYPRESS	n	9	9	9	16	16	16	16	16	13	9	10	10	14		14	13	8	7	6
CREEK B	$\bar{x}$	.08	.004	.02	.046	.048	.89	.071	.032	1.3	24.4	5.4	2.8	171		162	2.7	6.6	17.6	240
5-CYPRESS	n						3	3		1	3									
CREEK S	$\bar{x}$						1.02	.020		.1	23									
1-TROUT CREEK	n	9	9	9	16	16	16	16	16	13	9	9	10			14	14	8	6	4
	$\bar{x}$	.11	.007	.03	.123	.132	1.03	.131	.072	3.3	22.9	5.3	3.6			168	8.7	7.2	17.9	140
4-WEST BRANCH	n	9	10	10	14	14	15	15	14	11	10	8	8	12		12	12	6	5	4
TROUT CREEK	$\bar{x}$	.08	.008	.05	.022	.019	1.23	.028	.016	1.4	22.3	2.3	1.8	144		146	1.9	6.8	18.2	100
THIRD ORDER STREAM-STORM EVENT SAMPLING																				
STATION NO.																				
4-WEST BRANCH	n	8	8	8	10		11	1	10	11	11				10					
TROUT CREEK	$\bar{x}$	2.29	.26	.75	.047		.50	.010	.043	1.7	15.5				23.8					
9-TAYLOR	n	6	6	6	8		9	1	8	9	9				8					
SLOUGH	$\bar{x}$	7.58	.31	1.22	.094		.45	.040	.033	2.1	18.1				12.6					
10-OUTFALL	n	9	8	8	10		11		11	11	11				9					
SLOUGH	$\bar{x}$	2.42	.16	.98	.070		.43		.019	1.9	22.8				10.9					
12-N.FINGER	n	5	5	5	6		7		6	7	7				6					
SLOUGH	$\bar{x}$	1.71	.17	.88	.035		.47		.197	2.5	19.4				4.0					
15-OUTFLOW TO	n	5	5	5	7		8		7	8	8				7					
POWERLINE	$\bar{x}$	1.68	.10	1.45	.038		.24		.028	2.3	15.8				5.6					

* mg/l
** mg/m^3
n number of samples
$\bar{x}$ mean
B Background
S Storm Event

DISCUSSION

The diluting effect of increased flow in lower order streams was quite evident for certain parameters such as the Coliforms, BOD_5, TOC and Color. The bacterial parameters (total coliform, fecal coliform and fecal streptococci) were used to indicate possible concentrations of human pathogenic organisms. One problem with these indicators is that they are present in the feces of all warm-blooded animals to a certain extent. Recent studies (Doran and Linn, 1979) have shown that when these bacterial indicators are present in storm runoff from grazed and ungrazed pasture their levels can naturally exceed the criteria of the Department of Environmental Regulation. Doran and Linn (1979) also state that the fecal coliform to fecal streptococcus ratios for humans is 4.3; for cattle and other livestock and poultry it is .104 to .421 while for rabbits, birds and mice it is 0.0008 to 0.043. Some mixed pollution was indicated upstream in the Hillsborough River with the majority of the other stations showing livestock and poultry bacterial sources. The lowest bacterial ratios occurred in storm runoff from basins where the wetland to rangeland ratio was approximately 2:1.

The denitrification of nitrate to atmospheric nitrogen occurs in areas like the swamps and third order streams of the property where there is little or no dissolved oxygen concentration, pH >5.5, and abundant sources of Carbon (Dierberg and Brezonik, 1976) as is indicated by second and third order stream TOC levels.

Hunt and Lee (1976) suggested that when there is shallow sheet flow over organic substrates (such as exists in the sloughs of the property) conditions would be favorable for nitrification and denitrification to take place simultaneously. Ammonia is oxidized to nitrate in the aerobic water column, and conditions are favorable for denitrification to nitrogen gas a mere 5 mm below the surface of the sediment. This might not only explain why nitrate and nitrite are progressively lower as one moves from first to third order streams but also why ammonia and total kjeldahl nitrogen are not higher. For these reasons and because of the probable rapid utilization of nitrates by plants, nitrate was significantly lower at stations 4 and 5 than at station 3 on the Hillsborough River. There were no significant differences for either TKN or NH_3 among any of the baseline station pairs.

Unlike nitrogen, phosphate enters the water primarily from the dissolution of geological deposits. Total phosphate includes phosphate that is available in the water for plant use while the organically bound phosphate has already been incorporated and requires bacterial decomposition to be reutilized. As mentioned earlier, the Hillsborough River flows through a region that contains rich deposits of phosphate. For this reason and because of the heavy vegetative growth bordering third and second order streams the River had significantly higher concentrations of total phosphate than stations 2, 4 and 5. Total organic carbon concentrations were significantly higher at station 2 in Cypress Creek than at station 3 on the Hillsborough River.

Riverine swamps have the ability to effectively filter out suspended solids such as organic detritus. Larger litterfall, such as branches, twigs, etc. settle out when they become water-logged or caught in other vegetation. This is indicated in the storm event data which show lowest average suspended solids concentrations coming from basins with the lowest percentage of rangeland. The decomposition of these materials places an additional demand on the dissolved oxygen of the stream that is not measured by the BOD_5 test. For this reason diel measurements were performed at some stations permitting an overall determination of the community metabolism rates. This is not a precise method but it does show the order of magnitude of the respiration and production (Table 2). The BOD_5 values are less than twenty percent of any of the respiration rates and in most cases much less. Odum and Hoskins (1958) stated that if the dissolved oxygen concentration remains mostly below 100% saturation the community is heterotrophic, that is, it is re-

TABLE 2. COMMUNITY METABOLISM VALUES COMPUTED BY THE DIURNAL CURVE METHOD.

Stations	3/29-30/79		6/25-26/79	
	Respiration g0/m^3/day	Production g0/m^3/day	Respiration g0/m^3/day	Production g0/m^3/day
1			14.	7.3
2	2.4	.4	20.	1.4
3	53.	3.8	4.6	5.5
4	43.	3.1	11.	<.1
5	72.	12.	14.	5.6

ceiving more organic matter than it is producing. The Hillsborough River at station 3 was the only station that ever had values above 100% saturation. The baseline stations in second and third order streams and at station 3 on the Hillsborough River were heterotrophic tending toward dystrophy (Odum, 1971) because of the heavy litterfall in those types of swamps (Deghi, 1975). For these reasons the minimum dissolved oxygen concentrations were observed on occasion to be below the state criteria of 4 mg/l at all stations.

The pH of second order streams was lower than at the Hillsborough River stations. This difference is associated with the fact that the tannins that were leached into this water (cf. discussion of color) are weak organic acids which lower the pH in the absence of buffering carbonates.

ACKNOWLEDGEMENTS

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IMPLEMENTATION OF STORM WATER MANAGEMENT MODELS:

PROS AND CONS OF STANDARDIZATION

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and
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INTRODUCTION

Replacement of traditional methods for the design of urban drainage systems, such as the Rational Method, by modelling techniques has been promoted in the last few years by several governmental agencies in both Canada and the United States.

A review of the state of the art in urban drainage in Canada, carried on under the direction of the first writer, indicated that 5 years ago only one Canadian municipality -- Toronto -- was using urban drainage models (1). To our knowledge, at least seventeen municipalities have in the meantime conducted one or several modelling studies. Most of them, however, continue to use the Rational Method, or other "simplified" techniques.

The review by Poertner of methods used for the design of storage facilities (2) offers examples of a variety of "simplified" techniques leading to triangular or trapezoidal hydrographs, or to mass curves derived from the Rational Method. SCS has also promoted simplified graphs giving peak flows for "flat" or "steep" slopes and tabular methods for hydrographs and routing (3). In contrast, drainage manuals developed in Canada for various municipalities (4,5), or for the design of airport drainage (6) usually recommend, for larger areas, specific urban hydrologic models recently tested by U.S. EPA and the Canadian Drainage sub-committee. Examples of models described in these manuals are SWMM, ILLUDAS and unit hydrograph models initially developed for rural hydrology studies, such as HYMO.

Several studies conducted with the Rational Method indicated that, for the same area, peak flows determined by various engineers varied because of arbitrary selections of the inlet time or runoff coefficient (1, 7, 8). One of the arguments for modelling was to eliminate the effects of this subjective selection. If similar tests were conducted today, discrepancies in peak flows and hydrographs would vary with models, parameters, design storms, and could eventually be more significant than in the era of the Rational Method.

Decisions regarding the choice of the method are in most cases left with the drainage engineers of the municipality or the consulting firms who, in turn, are influenced by available funds, pressures to cut down initial estimates, personal inclination, in-house capabilities and other factors.

Decisions taken as a result of modelling activities may affect significant investments. In the design of new storm sewers and relief work they bear on the safety against basement flooding. Problems like the effects of runoff increase may also result in litigation. Legal responsibilities of modellers and consequences of differences between various models should also be considered.

Under these circumstances, control of the quality of modelling in urban drainage represents a serious challenge. Is the profession ready for a large scale application of sophisticated methods? Are these preferrable to the simple, traditional techniques which are easy to understand and verify? Is it desirable to standardize the techniques or should their selection be left entirely to those in charge of a given project?

There are no general or clear cut answers to these questions. Options may vary with the importance of the job or according to non-technical aspects, such as the training of the staff in charge of the verification of drainage projects.

It is however a reasonable objective that studies within the same drainage jurisdiction (municipality) should be based on the same philosophy regarding modelling, and that drainage manuals or local regulations define the position of the jurisdiction with regard to some of these difficult questions.

The aim of this paper is to present for discussion by the SWM users group, some modelling aspects to be considered in the development of drainage manuals or local regulations, and to present some suggestions regarding a more uniform approach in modelling. The presentation is limited to the quantitative aspects of runoff control, but some of the principles may be extended to other areas of storm water management.

ONE OR SEVERAL MODELS?

It seems that regulatory agencies and drainage jurisdictions have followed two conflicting schools of thought for the implementation of modelling.

The first one, considers that modelling is an art and should not be "regulated". Some governmental organizations seem to have made a conscientious effort to avoid sponsoring of a particular model, in order to stimulate research. This avoids criticism from model-builders or from consultants more conversant with a given model.

The second school of thought, typical for some flood control or erosion control agencies, is to require all those involved in a given type of study to consider the same procedures in some cases with as simple a method as possible.

The thesis of freedom of choice between "tested" models seemed to be supported by the results of various studies commissioned for the testing and comparison of models which did not result in clear cut advantages for one given model (7, 9, 10). Comparison of measurements on a number of small urban watersheds with simulated flows shows that models such as SWMM and ILLUDAS give an acceptable prediction within an error in the order of 20% (1).

These tests however were carried on by specialized hydrologists with a very careful selection of model parameters. Model comparisons have analyzed mainly real, frequent storms. Data on rare, very intense storms, which are of interest for design purposes, are practically not available. Typical differences between model comparisons, given in Figure 1(1), are not significant for the practitioner.

In routine design applications however, differences under design conditions may be significant and, would more likely resemble those in Figure 2, which shows a comparison of ILLUDAS, HYMO, and SWM models for conditions typical for runoff control studies in small watersheds. Differences of this nature are not considered acceptable. By reviewing the parameters in terms of physical data, results in this example could be brought closer. This however would require specialized hydrologic work which is usually not carried on in routine projects.

The direction favouring free selection of models by the hydrologist may be welcomed by scientists and specialized consultants. The argument for its implementation is that municipal engineers use specialized assistance and that modelling should become a speciality in urban drainage.

Real life, however, shows that this is not the case because most organizations prefer to conduct studies with existing in-house staff and will select models accordingly. Projects with sophisticated models are difficult to verify, modify and improve once the initial modeller is no more available. The fact that economic advantages of sophisticated modelling have not yet been clearly demonstrated, combined with the high cost of training, have encouraged many jurisdictions to sponsor standardization of simpler techniques.

Review of these policies indicates on the other hand that this approach does not only discourage the use of hydrologic expertise but may also give decision makers limited, if not biased information. It is therefore suggested that drainage jurisdictions should start with an analysis of their objectives and required information, and not with preselection of one or several models. The next section will give several examples in this direction.

SELECTION OF MODELS IN TERMS OF OBJECTIVES

Most SWM studies are based on surrogate objectives such as "zero runoff increase at the outlet of a new development". For real objectives, such as protection against flooding or erosion, it is important to have information on flow changes not only at the outlet but also along the downstream channel. If this viewpoint is accepted, the drainage jurisdiction has to recommend modelling on a watershed basis. The next logical step is that selected models need proper routing routines. Consideration of simultaneous operation of all storage facilities results from the requirement of a watershed approach.

The use of "zero runoff increase" regulation also requires that the agency clarify its position regarding the concept of "predevelopment flow". Drainage manuals or regulations do not consider the fact that predevelopment and postdevelopment flows with the same frequency will not occur for the same storm.

Another problem to be considered is that some urban models such as SWMM have not been extensively tested for rural conditions. Other models like HYMO have not been tested for urban conditions. A drainage jurisdiction in charge with runoff control may therefore consider two different models, or preferably conduct in-depth studies for the determination of pre-development flows on a wide area and recommend runoff rates for all those involved in development projects.

Storm sewers are designed for free surface flow for a "design frequency" which is also one of the current surrogate goals. According to the characteristics of the system, a storm with a higher return frequency may be conveyed with various surcharges. Protection against basement flooding, which is the real goal, will vary within the same jurisdiction. A requirement closer to the real objective should therefore be the limitation of surcharges. This would require a model which can handle routing under such surcharged conditions and the choice of the drainage engineer would be narrowed down to two or three models. The City of Toronto, for example, has based the design of its relief sewers on surcharge analysis determined by the HVM Model. Most SWM studies are conducted for one event models with design storms. Limitations of these procedures have been accepted as a result of cost and time constraints. A complete information on the response of various facilities is given by continuous simulation. The choice was made by several jurisdictions in Virginia, Maryland, and others who conduct SWM studies on the basis of the HSP model.

An intermediate solution is the use of several real storms, screened by means of the STORM model. This solution was adopted by the Metropolitan Toronto Conservation Authority for the overall hydrology of its large area (11).

Even if a jurisdiction works on the basis of single event - a design storm for each return frequency, the limitations should be determined by inlet studies. Consideration should be given at least in some projects to the following aspects which are not included in many drainage manuals or ordinances:

- sequences of storm events
- duration of "critical"storm event
- underestimation of rural flows and overestimating

storage by using storms which are critical for urbanized conditions

- requirements for simulation of snowmelt and frozen soil conditions
- effects of change in downstream timing of peak flows

A comprehensive analysis of resulting information against cost of modelling was presented for several watersheds by the first writer, for the Metropolitan Toronto Conservation Authority. Results indicated that the traditional one-storm and one-unit hydrograph model was estimated at approximately $60K, while an extensive long term simulation would cost approximately $120K. Based also on other considerations including time, available data for calibration and available experience of the jurisdiction, a multi-event simulation using STORM for screening of events with an estimated cost of $95K was finally selected. The point of this comparison was that the increase in cost for a more comprehensive analysis does not justify *on a priority* rejection. Even if jurisdictions may not have the additional funds required for in-depth studies, a group of agencies could at least conduct demonstration projects with various models. For most large developers, an in-depth study may not only represent a negligible additional expense, but could also result in savings by eliminating facilities required by a simplistic approach required to meet the surrogate goals. If regulations for "floodline determinations" and "zero runoff increase" are simplistic or rather arbitrary, advanced modelling becomes a fallacy. Promotion of better modelling should therefore start with the improvement of the decision making process.

RESPONSIBILITIES IN MODELLING STUDIES

Responsibilities in the selection and application of modelling should be defined more specifically for the different groups participating in the storm water management planning and design processes. The drainage manuals reviewed by the authors do not deal with this delicate question.

The responsibilities of the jurisdiction in charge with the implementation of the results of storm water management studies is obviously to define the required information from a modelling process in order to select the best alternatives. As discussed in the previous section, presently used surrogate objectives do not coincide with this kind of information and may be even misleading. On the other hand, their use is simple and verification of projects carried out on this basis may become a routine operation.

The nature of the information required to meet the real objectives may not be very explicit at the beginning of a SWM study. For these reasons, a staged approach may help the jurisdiction in defining study objectives. Preliminary SWM regulations of the Town of Markham have been developed on this basis.

The first stage of a study has to define the main restrictions, needs for runoff control, and will compare broad alternatives. In this stage, the selection of models does not affect directly the investments and is left with the consultant. In the second stage, the choice of a model is derived from the conclusions and type of alternatives selected in Stage I. For example, if dual storage is required, a sophisticated model such as SWM with the WRE TRANSPORT routine as discussed in another paper (12) will be one of the best choices. For a smaller area without controls, the Rational Method will be sufficient.

The responsibility of those in charge of the design of SWM facilities is of course, to select a model which has the algorithm appropriate for giving the required information.

Model selection is quite often guided by available facilities from governmental agencies, computer companies, or specialized consultants. The responsibility of those providing the software has not yet been defined in SWM studies. In structural engineering however, it is considered that the developers of the software have the responsibility of providing models which reproduce correctly the assumptions in the algorithm (13). At the present time, those distributing or marketing SWM software are not penalized in user problems resulting from bugs, instability and perhaps incorrect assumptions. For some proprietory models, detailed algorithms are not available. Drainage manuals may require limitation in this regard. Those conducting modelling studies should also provide, for example, references for testing and previous applications. The selected model ideally should be calibrated for the same or similar conditions and have at least a good documentation with a presentation of its limitations.

IMMEDIATE STANDARDIZATION NEEDS

Once a model has been selected on the basis of the previous considerations, a number of factors related to its usage should be uniformized within a given jurisdiction. Some of the models have several generations with various assumptions or with different parameters. On the other hand, results from some models are very sensitive to schematization of the watershed or selection of characteristics of the system.

These factors are not discussed in most of the existing drainage manuals. As a result studies with "the same model" may lead to different results. Examples of modelling features which require immediate attention are:

(a) For the HYMO model, the relation between the parameters K and T_p. Several relations presently used in Ontario and corresponding results are given in Figure 3.

(b) The RUNOFF model in SWMM is affected by selection of the "width" parameter. On the other hand, there are two different schools of thought with regard to the selection of the width indicated in Figure 4.

(c) The SWMM TRANSPORT models have been used with various assumptions for the surcharge conditions. Some of them lead to unrealistic storages at the nodes with the result of underestimating the flows with surcharged conditions. Recent versions have reduced or eliminated these problems (14).

(d) For some models, if the level of discretization or lumping is done arbitrarily, the effects may be significant (9) (Figure 5).

(e) The distribution of the design storm has recently been given increased attention. What is not always considered is that some models are oversensitive to the discretization of the design storm, e.g. the RUNOFF model may lead to different flows for different levels of design storm discretization (15).

(f) The ILLUDAS model has now several routines for routing. Selection of the appropriate routine has to be made in terms of the nature of the problem.

FINAL REMARKS: THE IMPSM PROJECT

The list in the previous section is not exhaustive. Several papers and reports have already dealt with the sensitivity analysis and made suggestions regarding these elements (9, 16, 17). These results must be organized, tested again on the same basis and accepted by those who work for the same drainage jurisdiction. While models will be continuously improved, it is suggested that these improvements be incorporated in current practice in an organized fashion, with testing and users consensus at each step.

Until test data become available, a cautious approach when applying new or more sophisticated procedures is required. The time has come to take advantage of collective past experience in practical modelling, including mistakes, a step towards the uniformization of modelling procedures has recently started at the University of Ottawa. A group of governmental agencies, consultants and municipalities cooperate in the IMPSWM program (which stands for the "Implementation of Storm Water Management Models") of the University of Ottawa directed by the authors has started after 6 months of discussions with various direct and indirect model users. The philosophy is that, at present, standardization of the factors discussed in the previous section and recommendations regarding simplified techniques may start by a consensus of users, based on discussion of research results. It is also hoped that IMPSWM participants will use the same programmes updated on a regular basis and will give the whole group information on good and bad experiences. IMPSWM will also organize training activities for a broad audience and for users for discussion of specialized problems with a small group of specialized participants. Most participants are from Canada, but we have established a cooperation with U.S. specialists and already have received significant assistance from Dr. Larry Roesner from CDM - Water Resources Engineers. Several contacts have also been established in the U.K. and Switzerland.

This may be a modest step only but it is an old engineering experience that once a problem is recognized, the solution is very close. It seems from our discussions with consultants, municipalities and various agencies that the present confusion in modelling procedures is not perceived as a problem by some direct or indirect users. The principal aim of this presentation is to stimulate discussions on these aspects and to recommend the striking of a balance between scientific requirements for continuous improvements and implementation realities.

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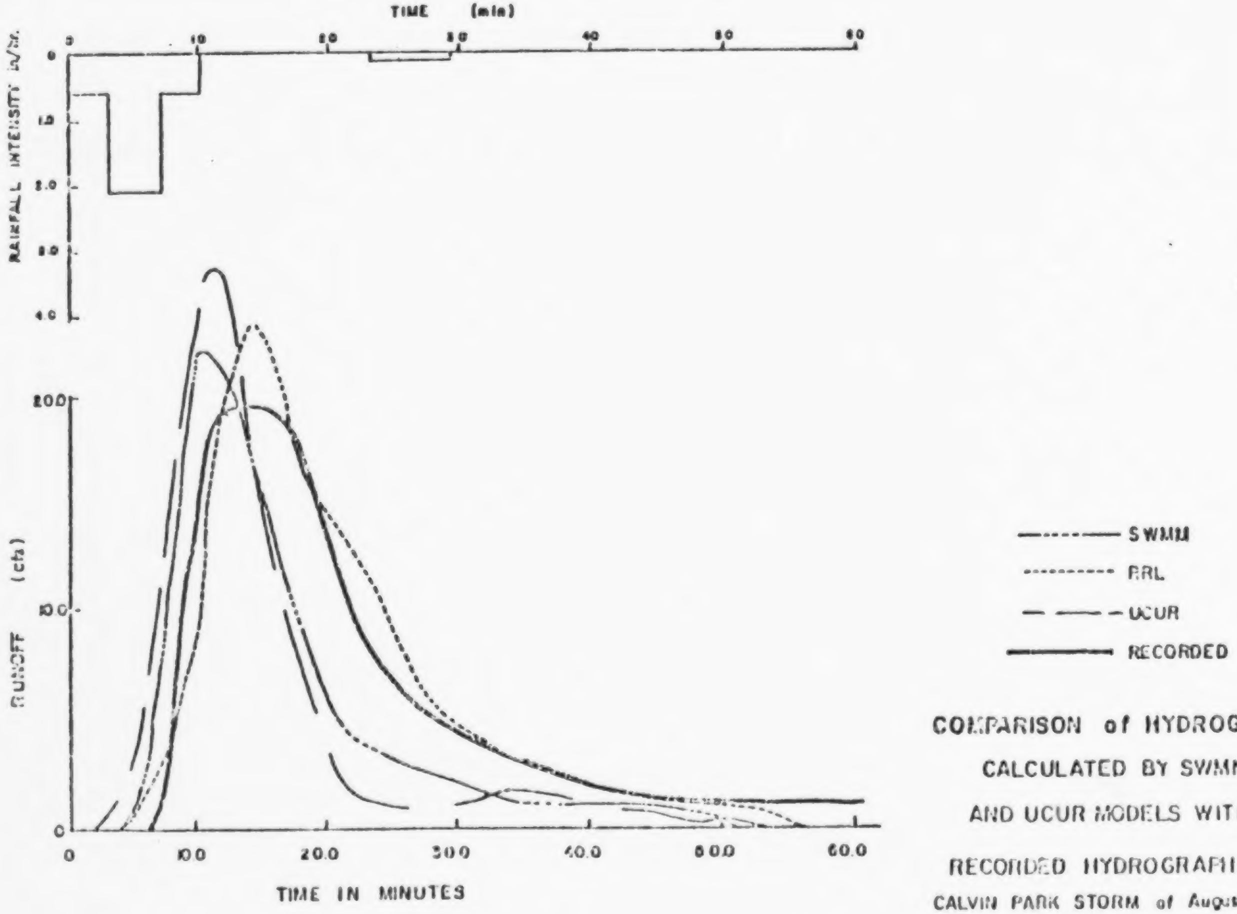

COMPARISON of HYDROGRAPHS
CALCULATED BY SWMM, RRL
AND UCUR MODELS WITH
RECORDED HYDROGRAPH
CALVIN PARK STORM of August 23, 1972

FIG 1

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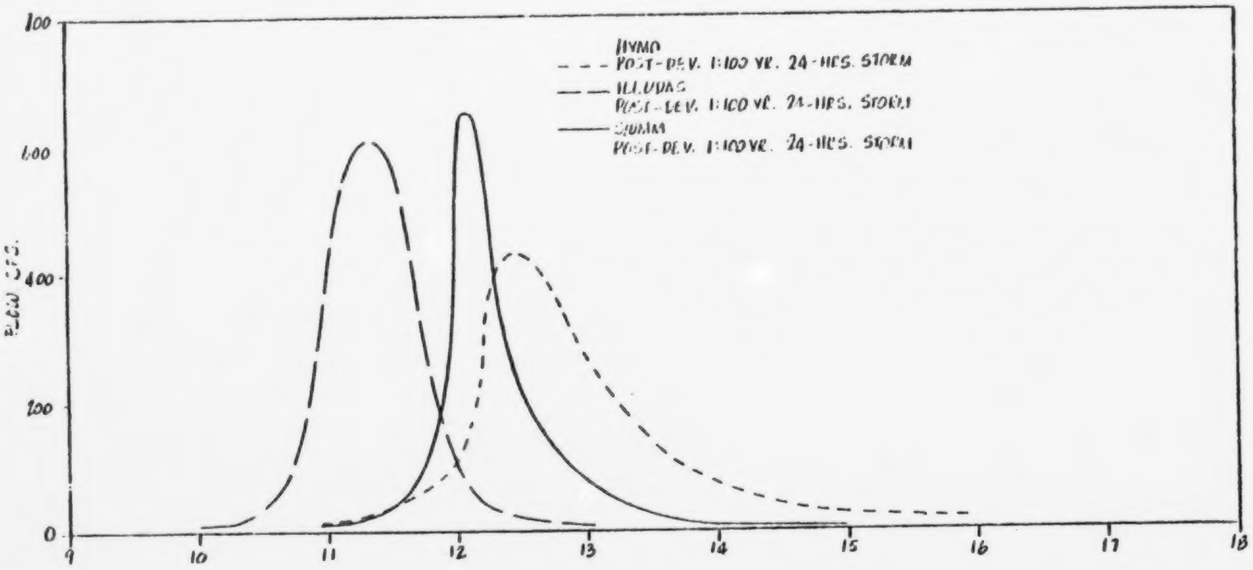

FIG. 2 - DIFFERENCES BETWEEN MODELS UNDER DESIGN CONDITIONS

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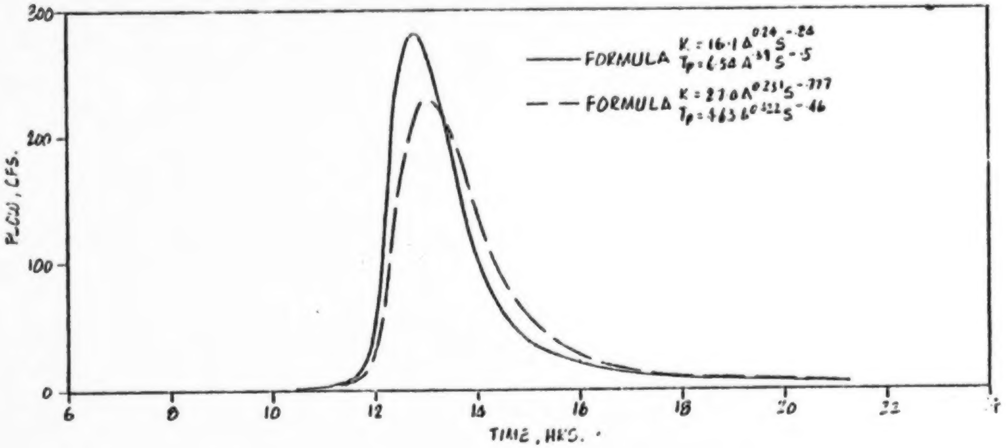
best available copy.

FIGURE 3

COMPARISON OF OUTFALL HYDROGRAPHS COMPUTED BY TWO TYPES OF FORMULAS FOR 'K' AND 'T_p' IN THE 'HYMO' MODEL.



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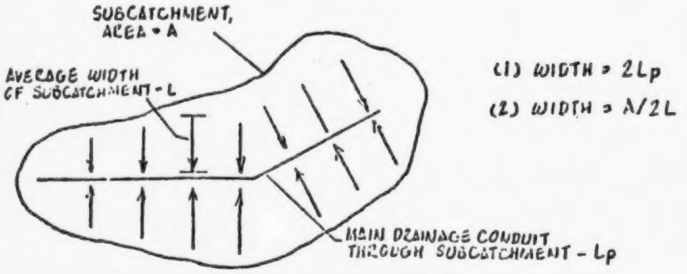
SUBCATCHMENT,
AREA = A
AVERAGE WIDTH
OF SUBCATCHMENT - L
(1) WIDTH = 2Lp
(2) WIDTH = A/2L
MAIN DRAINAGE CONDUIT
THROUGH SUBCATCHMENT - Lp

FIGURE 4 - Selection of Width parameter in runoff model.

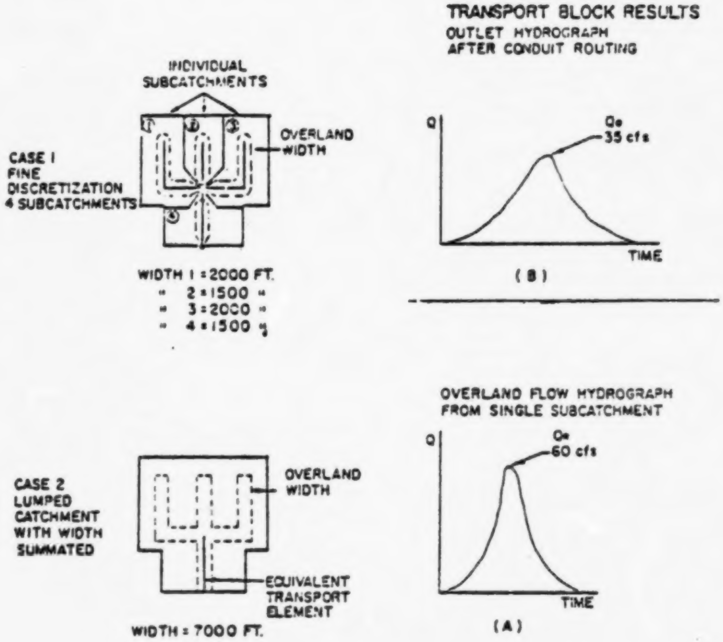
TRANSPORT BLOCK RESULTS
OUTLET HYDROGRAPH
AFTER CONDUIT ROUTING
INDIVIDUAL SUBCATCHMENTS
1
2
3
4
OVERLAND WIDTH
CASE 1
FINE
DISCRETIZATION
4 SUBCATCHMENTS
Q
Qp
35 cfs
TIME
(B)
WIDTH 1 = 2000 FT.
" 2 = 1500 "
" 3 = 2000 "
" 4 = 1500 "
OVERLAND FLOW HYDROGRAPH
FROM SINGLE SUBCATCHMENT
Q
Qp
60 cfs
CASE 2
LUMPED
CATCHMENT
WITH WIDTH
SUMMATED
OVERLAND WIDTH
EQUIVALENT
TRANSPORT
ELEMENT
WIDTH = 7000 FT.
TIME
(A)

FIGURE 5

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January 10-11, 1980

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